APPENDIX C
PRELIMINARY CHANNEL STABILITY ASSESSMENT

C.1 CHANNEL STABILITY APPROACH AND METHODS

Several approaches exist for developing design criteria and evaluating channel stability for river reconstruction efforts. Approaches are generally categorized as analog, empirical or analytical methods. Analog methods rely on field measurements from stable reference reaches to develop dimensionless ratios for pattern, cross section and profile characteristics that are applied to design reaches. Empirical methods use regime equations and regional hydraulic geometry relationships to predict hydraulic properties. Analytical methods employ numerical models to simulate hydraulic conditions for a range input variables. Due to theoretical assumptions and limitations that are unique to each method, an unacceptable range of answers typically results. Therefore, the standard channel stability assessment involves using a combination of several methods and relying on experience, practicality and judgment to interpret the results.

In the absence of a standardized approach to stream channel design, the design team used a combination of elements from several techniques that represent the best available methods for developing design criteria for restoration designs. Interpreting results, measuring channel stability and establishing acceptable design thresholds relied on the experience and judgment of the designers, the application of standard hydraulic principles and not necessarily an established or accepted set of design criteria. As described in Section 3, analog, empirical, and analytical methods provided the basis for developing a range of design channel dimensions, and were used to predict the most probable cross section, plan form and profile dimensions for the CFR and BFR.

Final channel dimensions were established through a trial and error process that included several iterations of channel stability analyses. The methods used to complete the channel stability analysis focused on refining the cross section, plan form and profile dimensions until acceptable values were observed for each hydraulic parameter considered to influence the dynamic equilibrium of river hydraulics. Acceptable values were assumed to be a range of values that best achieved dynamic equilibrium among hydraulic parameters, anticipated sediment size and anticipated sediment supply. Emphasis was placed on the results of analog methods for the selection of cross section, plan form and profile dimensions with considerable consideration given to sediment supply and sediment size. A range of +/-15% around a mean value for a specific hydraulic parameter was considered to be the guideline for the variation that a newly constructed river can accommodate before dynamic equilibrium is disrupted and instability is triggered. Although a statistical analysis was not employed, this guideline, along with additional consideration of reference reach conditions, was employed as threshold design criteria for channel stability and maintaining dynamic equilibrium. For reporting purposes, only the final iteration of analyses was reported in Appendix C – Preliminary Channel Stability Assessment.

In terms of the analyses in Appendix C, channel stability represents a condition where several hydraulic variables are balanced to achieve a state of dynamic equilibrium that approximates reference conditions, satisfies traditional hydraulic design principles and considers results from the best available regime equations. Channel stability implies that although the channel pattern may change over time, the channel's cross section area and slope remain consistent. Under stable conditions, rates of erosion and deposition are approximately balanced as the channel pattern changes.

The scale of this project along with the public health risks and implications of failure on surrounding infrastructure resulted in an intensive data collection effort that allowed the use of several methods to evaluate channel stability and predict stable, post-dam-removal river and floodplain conditions. As discussed in Appendix B, significant effort was devoted to data collection on selected reference reaches on the CFR and BFR both upstream and downstream of the restoration project area. These reaches are assumed to be stable and functioning in a state of dynamic equilibrium. In addition to displaying stability, the reference reaches possess the desired biological environment for aquatic wildlife and riparian vegetation species. For these reasons, emphasis was placed on methods that derived results from reference reach data. Figure C-1 identifies the location of the selected reference reaches in relation to the restoration project area. Sheet I-7 in Appendix I identifies the locations of reference reaches in CFR3.

The BFR Gage, CFR3 Middle (CFR3-B, see Figure 1-1) and CFR Bandmann Flats reference sites were identified as best examples of the most probable channel conditions for the BFR1 reach, CFR2/CFR3 reach and CFR1 reach, respectively. To validate the assumption that the selected reference reaches are stable, a comparison of hydraulic properties predicted by methods using analog, empirical and analytical approaches was completed. Results were used to establish a range of design criteria for proposed cross section and profile dimensions for the BFR and CFR.

The following assessments focus on validating the feasibility of the conceptual plan. As such, assessments were based on conceptual design layouts. Specific hydraulic properties evaluated included slope, velocity, shear stress, width-depth ratio, and sediment transport (power). Hydraulic properties were computed for a range of discharges from bankfull discharge to 100-year discharge. Additional consideration was given to effects of ice floes and ice jams on channel stability. An assessment of the selected channel dimensions over a range of discharges was completed using HECRAS and is presented at the end of this appendix. Hydraulic modeling efforts undertaken for proposed conditions used simplified channel cross sections and floodplain geometry to estimate potential design conditions. Hydraulic parameters derived from the analysis were used as input for calculations presented in the channel stability assessment. Further refinement of these assessments is expected during the final design phase.

C.2 STABLE SLOPE ANALYSIS

Formation of equilibrium channel slope is influenced by valley longitudinal slope, extent of lateral channel confinement within the valley and substrate composition. Generally, methods for estimating stable channel slope predict slopes that correspond to a threshold hydraulic condition for the initiation of movement for a specific bed material size class. As such, available equations rely on input data for bed composition, mean depth and dominant discharge.

C.2.1 Methods

A stable slope analysis was completed for selected reference reaches. Estimated slopes were compared with actual average slopes and used to validate proposed channel slopes. For the purpose of this analysis, it was assumed that available bed material in the restoration project area will resemble that found in the corresponding reference reaches. The following five methods were selected to evaluate critical slope.

Schoklitsch (1932)
$S = K\,(D_{50}/Q)^{0.75}$
S = Slope (ft/ft) — D_{50} = Particle size for which 50% is finer (mm)
K = Constant, 0.00174 — Q = Dominant discharge (cfs)

Meyer-Peter and Müller (1948)
$S = [K\,(Q/Q_b)(n_s/(D_{90}{}^{1/6}))^{3/2}D_{50}]/d$
S = Slope (ft/ft) — Q = Total discharge (cfs)
K = Constant, 0.19 — Q_b = Discharge over bed of river (cfs)
n_s = Manning's roughness for bed — D_{90} = Particle size for which 90% is finer (mm)
D_{50} = Particle size for which 50% is finer (mm)

MacBroom (1981)
$S = (KW_{bf}D_{50}/Q)^{0.75}$
S = Slope (ft/ft) — D_{50} = Particle size for which 50% is finer(mm)
K = Constant, 0.00021 — Q = Dominant discharge (cfs)
W_{bf} = Bankfull width (ft)

Rosgen (2001)
$S = \tau^*_c\,\gamma_s\,D_i/D$ S = Slope (ft/ft) — τ^*_c = critical dimensionless shear stress
D = Mean depth (ft) — D_i = Pavement D_{100} or bar sample D_{100} (ft)
γ_s = Specific weight of sediment (lbs/ft^3)

Thorne and Hey (1997)
$S = 0.087Q^{-0.43}D_{50}{}^{-0.09}D_{84}{}^{0.84}Q_s{}^{0.10}$
S = Slope (m/m) — D_{50} = Particle size for which 50% is finer (mm)
Q = Bankfull discharge (cms) — D_{84} = Particle size for which 84% is finer (mm)
Q_s = Sediment discharge (kg/m)

US Army Corps of Engineers (1994) Nomograph

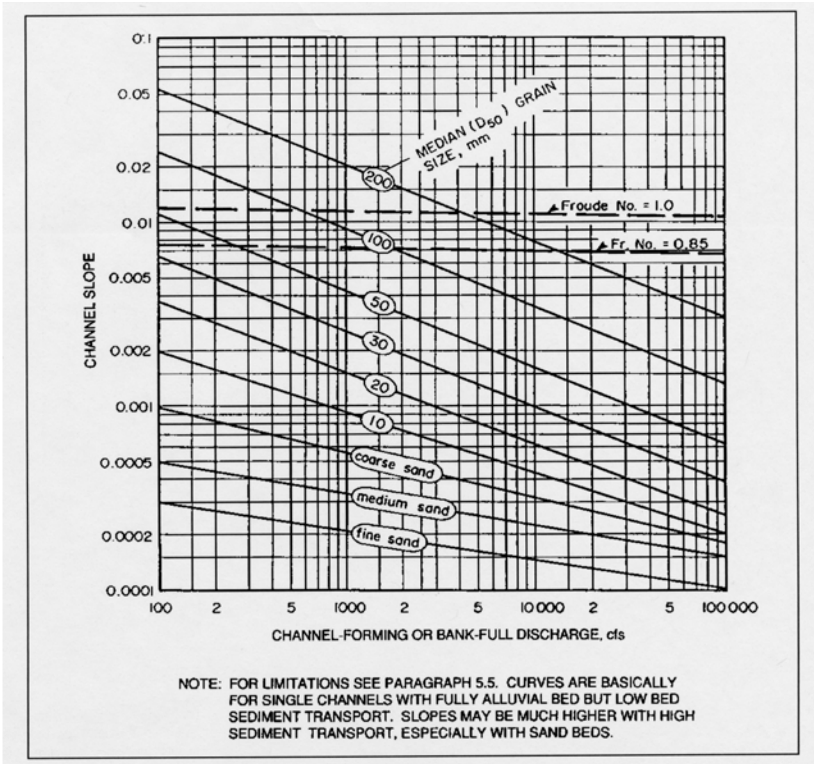


Figure C-2. Guide to slope-discharge relationships for erodible channels.

As with other empirical methods, these equations have limitations of applicability. Many have been developed for the purpose of designing armored flood control channels. Also, some equations incorporate factors of safety to ensure that bed movement does not occur. In addition, some equations have been calibrated to uniform particle sizes and sand bed channels. For these reasons, a wide range of results can be expected.

C.2.2 Results

Table C-1 presents the results of the analysis.

Table C-1. Comparison of estimated stable slope and average slope (ft/ft) for selected reference reaches and proposed reaches.

Critical Slope Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Schoklitsch	0.0033	0.0028	0.0049	0.0029	0.0037	0.0027
Meyer-Peter and Muller	0.0047	0.0044	0.0098	0.0069	0.0092	0.0067
US Army Corps of Engineers	0.0027	0.0020	0.0055	0.0030	0.0035	0.00250
MacBroom	0.0033	0.0028	0.0049	0.0029	0.0037	0.0027
Rosgen	0.0035	0.0033	0.0063	0.0048	0.0056	0.0041
Thorn and Hey	0.0043	0.0044	0.0064	0.0043	0.0045	0.0041
Average of Methods	0.0040	0.0034	0.0061	0.0039	0.0047	0.0042
Actual Average Slope	0.0028	0.0019	0.0032	0.0024	0.0025	0.0024
Selected Design Slope	0.0031 - 0.0043	0.0030 - 0.0032	0.0025 - 0.0040	N/A	N/A	N/A

C.2.3 Conclusions

Results indicate that actual average slopes are less than estimated stable slopes for nearly all methods and all reaches. Since the selected reference reaches are stable and do not appear to be aggrading, it can be concluded that a range of acceptable slopes is available for the proposed design reaches.

Existing project constraints and required tie-in points throughout the restoration project area dictated the values for selected design slopes in Table C-1. Despite these limitations, most selected design slopes lie within the acceptable range for stable slope. For those values less than the acceptable range, it may be necessary to adjust other hydraulic variables such as width-depth ratio so that the proposed channel does not experience aggradation. For those values greater than the acceptable range, it may be necessary to increase the frequency of grade control structures or increase the width-depth ratio.

C.3 VELOCITY ASSESSMENT

Channel velocity is a function of channel slope, channel geometry and channel roughness. Available methods for estimating critical velocity predict mean velocities that correspond to a threshold hydraulic condition for the initiation of movement for a specific bed material size class. As such, available equations rely on input data for bed composition, mean depth and slope.

C.3.1 Methods

A critical velocity analysis was completed for selected reference reaches. A combination of analog, empirical and analytical methods was used. Hydraulic models used included WinXSPro (WEST Consultants, Inc. 1998) and HEC-RAS (US Army Corps of Engineers 2004). Both models were calibrated by using field-surveyed channel geometry, bankfull indicators and bankfull discharges to predict Manning's roughness values. Table C-2 presents a summary of the Manning's roughness values used for the analysis.

Table C-2. Summary of estimated Manning's roughness values for selected reference reaches and proposed reaches at bankfull discharge.

Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
HEC-RAS	0.031 (0.025–0.036)	0.032 (0.027–0.037)	0.039 (0.038-0.044)	0.032 (0.028-0.036)	0.036 (0.034-0.038)	0.035 (0.032-0.038)
Hey	0.035 (0.034–0.037)	0.037 (0.036–0.038)	0.043 (0.042-0.044)	0.038 (0.037-0.039)	0.046 (0.042-0.049)	0.037 (0.037-0.038)
WinXSPro	0.029 (0.025–0.036)	0.032 (0.027–0.037)	0.035 (0.032-0.039)	0.032 (0.028-0.036)	0.036 (0.034-0.038)	0.035 (0.032-0.038)
Average of Methods	0.032	0.034	0.039	0.036	0.039	0.036
Selected Design Roughness	0.032-0.035	0.040	0.037	N/A	N/A	N/A

In addition to estimated roughness values presented in Table C-2, bankfull roughness values were calculated for the BFR Bonner gage and CFR Missoula gage. WinXSPro was used to process field surveyed bankfull indicators and hydraulic geometry, and calibrate roughness values. Calculated values were 0.039 for the BFR Bonner gage and 0.030 for the CFR Missoula gage.

The following five methods were selected to evaluate velocity.

<u>Isbash (1936)</u>
$V = 10.7\ (D_{50})^{1/3}\ d^{1/6}$
V = Permissible velocity (ft/s) D_{50} = Particle size for which 50% is finer (ft)
d = Mean depth (ft)

<u>Mavis and Laushey (1948)</u>
$D_c = 1.88\ V_m^{\ 2}$
V = Mean velocity (ft/s) D_c = Armor size (mm)

Hey (1979)
Modified Darcy-Weisbach
$U = (8/f)^{1/2 =}$ 5.75 $\log(aR/3.5*D_{84})$
$a = 11.1(R/d_m)^{-0.314}$
$u* = (9.8RS)^{\wedge 0.5}$
f = Darcy-Weisbach resistance coefficient
a = Function of channel cross-sectional shape varying between 11.1 and 13.46
R = Hydraulic radius (m)
D_{84} = Particle size for which 84% is finer (m)
d_m = Maximum depth at the section (m)
U = Mean velocity (m/s)
u_* = Mean shear velocity (m/s)

Manning (1891)
$V = [1.49R^{2/3}S^{1/2}]/n$
V = Mean velocity (ft/s) R = Hydraulic radius (ft)
S = Slope (ft/ft) n = Manning's resistance coefficient

Estimated critical velocities were used to validate proposed channel velocities and compute proposed channel cross sectional area. For the purpose of this analysis, it was assumed that available bed material in the restoration project area will resemble that found in the corresponding reference reaches. As with other empirical methods, these equations have limitations of applicability. Many have been developed for the purpose of designing armored flood control channels. Also, some equations incorporate factors of safety to ensure that bed movement does not occur. In addition, some equations have been calibrated to uniform particle sizes and sand bed channels. For these reasons, a wide range of results can be expected.

C.3.2 Results

Table C-3 presents the results of the analysis.

Table C-3. Comparison of critical velocity and calculated velocity (ft/s) for selected reference reaches and proposed reaches.

Velocity Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Isbash	5.1 - 7.1	5.8 - 9.0	8.0 - 10.6	5.6- 8.0	6.3 - 9.6	5.2 - 7.9
Mavis and Laushey	5.2 - 7.3	5.9 - 9.2	8.3 - 10.9	5.7- 8.2	6.4 - 9.9	5.4 - 8.1
Hey	5.1	6.0	5.4	4.0	3.6	4.7
Manning (Q_{bf})	5.7	7.2	6.6	4.9	4.6	5.0
Manning (Q_{bf})	5.5	7.1	6.1	5.4	5.2	5.3
Manning (Q_2)	6.2	8.1	6.9	6.3	5.9	6.2

Table C-3 (Continued). Comparison of critical velocity and calculated velocity (ft/s) for selected reference reaches and proposed reaches.

Velocity Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Manning (Q_{10})	7.8	9.8	8.2	6.8	6.4	7.1
Manning (Q_{25})	8.5	10.4	8.7	7.2	6.8	7.7
Manning (Q_{50})	8.9	10.8	9.0	7.8	7.1	8.5
Manning (Q_{100})	9.3	11.1	9.2	8.4	7.3	9.1
Manning (Q_{500})	10.2	11.5	9.7	10.3	7.4	11.0
Average of Methods (Q_{bf})	5.3	6.4	6.9	5.1	5.2	5.1
Selected Design Velocity (Q_{bf})	5.8	7.0	6.4	N/A	N/A	N/A

* Critical velocity range calculated for D_{50} and D_{84} particle sizes.

In addition to velocity values presented in Table C-3, velocity values were calculated for the BFR Bonner gage and CFR Missoula gage. WinXSPro was used to process field surveyed bankfull indicators and hydraulic geometry, and calibrate roughness values. Calculated values were 6.6 ft/s for the BFR Bonner gage and 5.9 ft/s for the CFR Missoula gage.

C.3.3 Conclusions

Results yielded an acceptable range of velocities for bankfull discharge conditions. As discussed in C.2, existing project constraints and required tie-in points throughout the restoration project area dictated the values for selected design slopes. Variations in slope between reference reaches and project reaches account for the difference between average calculated mean velocity and selected design mean velocity.

Despite these limitations, selected design velocities lie within the acceptable range for mean velocity. For those values less than the acceptable range, it may be necessary to adjust other hydraulic variables such as width-depth ratio so that the proposed channel does not experience aggradation. For those values greater than the acceptable range, it may be necessary to increase channel roughness with larger substrate or increase the width-depth ratio.

C.4 HYDRAULIC GEOMETRY ASSESSMENT

The fundamental concept behind assessing stable channel geometry is to establish channel dimensions that function in dynamic equilibrium. Cross section geometry, in particular, is influenced by many variables including valley morphology, channel forming discharge, slope, sinuosity, sediment supply and sediment composition. There have been extensive efforts directed toward the development of hydraulic geometry regime equations for gravel bed rivers. Most regime equations predict geometry as a function of channel forming discharge. However, regime equations typically produce a wide range of results due to the difference in experimental conditions under which they were developed. As an alternative, data from stable reference reaches is another useful means for predicting stable channel geometry. Other methods have been developed to predict stable channel dimensions for flood control channels with engineered linings.

C.4.1 Methods

A hydraulic geometry assessment was completed for the selected reference reaches and proposed project reaches. The assessment compared results predicted by applicable regime equations with measured values from selected reference reaches. Average results were used as guidance for developing width-depth ratios for proposed channel cross sections. Efforts focused on developing stable riffle cross section dimensions. The following regime equations were selected for the analysis:

Williams (1986)

$W = (Q_{bf}/0.4)^{0.55}$ $D = 0.12W^{0.69}$

W = Bankfull width (ft) Q_{bf} = Bankfull discharge (cfs) D = Mean depth (ft)

Millar (2004)

$W = 16.5Q^{*0.70}S^{0.60}\mu'^{-1.10}$ $D = 0.125Q^{*0.16}S^{-0.62}\mu'^{0.64}$

W = Bankfull width (m) D = Mean depth (m)

S = Slope (m/m) D_{50} = Particle size for which 50% is finer (m)

Q* = Dimensionless discharge = $Q/[D^2_{50}\sqrt{(gD_{50}(s-1))}]$

g = gravitational acceleration (m/s^2)

s = specific weight of sediment (N/m^3)

μ'= ratio of critical bank shear stress to critical bed shear stress (1.0 for no bank vegetation, 1.4 for established bank vegetation)

Hey and Thorne (1979)

$W = 4.33Q_{bf}^{0.55}$ $D = 0.22\ Q_{bf}^{0.37}D_{50}^{-0.11}$

W = Bankfull width (m) Q_{bf} = Bankfull discharge (cms)

D = Mean depth (m) D_{50} = Particle size for which 50% is finer (m)

Rosgen (2001)

$D = \tau^*_c\ \gamma_s\ D_i/S$

D = Mean depth (ft) τ^*_c = critical dimensionless shear stress

S = Slope (ft/ft) D_i = Pavement D_{100} or bar sample D_{100} (ft)

γ_s = Specific weight of sediment (lbs/ft^3)

Bray (1982)

$W = 2.38Q_2^{0.53}$ $D = 0.226\ Q_2^{0.33}$

W = Bankfull width (ft) Q_{bf} = Bankfull discharge (cfs) D = Mean depth (ft)

Simons and Albertson (1963)

$W = 2.5Q_{bf}^{0.51}$	$D = 0.43W^{0.36}$	
W = Bankfull width (m)	Q_{bf} = Bankfull discharge (cms)	D = Mean depth (m)

Lacey (1948)

$W = 2.67Q_{bf}^{0.5}$	$D = Q_{bf}/[13.5\sqrt{(25.4D_{50})}]^{1/3}$	
W = Bankfull width (ft)	Q_{bf} = Bankfull discharge (cfs)	D = Mean depth (ft)

C.4.2 Results

Tabled C-4 and C-5 present the results of the analysis.

Table C-4. Comparison of width (ft) and mean depth (ft) for selected reference reaches and proposed reaches.

Geometry Method	CFR3 Middle C4			CFR1 Bandmann F3/1			BFR1 Bonner B3		
	W	D	W/D	W	D	W/D	W	D	W/D
Williams	140	3.6	39	268	5.7	47	202	4.7	43
Millar	154	3.7	41	260	5.9	44	110	8.0	14
Hey and Thorne	135	5.3	25	244	8.0	31	188	6.1	31
Rosgen		4.0			7.8			8.2	
Bray	208	3.7	57	383	5.3	72	291	4.5	65
Simons and Albertson	82	7.1	11	149	10.9	14	114	9.1	13
Lacey	151	4.9	31	272	6.9	40	210	5.2	40
Average of Methods	142	4.3	34	263	6.4	43	186	6.0	36
Average of Reference Reaches	141	3.1	45	238	6.2	38	200	5.1	39
Selected Average Design Value	150	3.7	40	245	6.1	40	200	4.9	40

Table C-5. Comparison of width (ft) and mean depth (ft) for selected reference reaches.

Reach	BFR Ovando C4			BFR Ovando B3			BFR Ovando F4		
Geometry Method	W	D	W/D	W	D	W/D	W	D	W/D
Williams	147	3.8	39	147	3.8	39	147	3.8	39
Millar	90	6.2	15	78	6.9	11	100	5.7	17
Hey and Thorne	141	5.4	26	141	5.2	27	141	5.5	26
Rosgen		5.8			6.9			5.4	
Bray	180	3.3	54	180	3.3	54	180	3.3	54
Simons and Albertson	107	4.9	22	107	4.9	22	107	4.9	22
Lacey	158	4.8	33	158	4.7	34	158	5.0	32
Average of Methods	137	4.9	28	132	5.1	26	139	4.8	29
Average of Reference Reaches	170	2.9	59	191	3.0	64	183	3.0	61

C.4.3 Conclusions

Results produced a wide range of values. As mentioned previously, differences in experimental conditions under which the equations were developed is a likely cause for the variability. However, the average of all methods when compared to actual reference reach values produced an acceptable range of values. When selecting design values, consideration was given to regime equations however, greater emphasis was placed on observed values from reference reaches.

C.5 SHEAR STRESS ANALYSIS

The incipient motion for a sediment particle by flowing water is a key concept in estimating shear stress and sediment transport. The movement of a single particle is influenced by a range of factors including hydraulic geometry, slope, bed composition and bed armoring. The force at which point a particular size particle is set in motion is termed the critical shear stress of that particle size. The Shields equation is used to calculate critical shear stress (Shields 1936).

$$\tau_c = \tau_c^* (\gamma_s - \gamma) D$$

where τ_c^* is the dimensionless Shields parameter for entrainment of a sediment particle of size D, and γ_s and γ are the unit weights of sediment and water, respectively, expressed in pounds per cubic foot. Generally, the parameter D is taken to be D_{50}, the median grain size of the bed sediment, and, dimensionally, must be in units of feet.

The Shields critical shear stress equation has been experimentally evaluated in laboratory flumes using uniform size particles. Experimental results suggest a constant value in the range of 0.04 to 0.06 (cited in Reid et al. 1997). Extensive work has been done to evaluate critical shear stress in natural channels characterized by a range of conditions (Montgomery and Buffington 1997).

Shear stress assessments typically compare critical shear stress to average total shear stress. Average shear stress is calculated by the equation,

$$\tau = \gamma R s$$

where τ is the shear stress, γ is the specific weight of water, R is the hydraulic radius, and s is the slope of the channel.

Concerning channel design methods, the engineered channel is typically designed to generate a critical shear stress capable of initiating motion in a D_{84} size particle during a bankfull discharge event. This concept is consistent with several studies (Pickup 1976; Jackson and Beschta 1992; Grant 1987; Carling 1988; Sidle 1998; Booth 1990; and Leopold 1992). Significantly increasing or decreasing stream energy by modifying channel dimensions may lead to channel degradation or aggradation, respectively.

C.5.1 Methods

A critical shear stress analysis was completed for selected reference reaches. Hydraulic models were developed using HEC-RAS (US Army Corps of Engineers 2004) to estimate average total shear stress. As indicated previously, the model was calibrated by using field-surveyed channel geometry, bankfull indicators and bankfull discharges to predict Manning's roughness values.

Most available methods for calculating critical shear stress use the Shields equation, but differ in their means of calculating critical dimensionless shear stress. There are numerous critical dimensionless shear stress equations that have been developed to predict incipient motion. Equations are generally derived from laboratory flume experiments that used sand bed channels to develop sediment transport relationships. Models based on field data collected in alluvial, gravel-bed rivers are less common. The following two methods were selected for their applicability to pool-riffle bed forms.

Parker and Klingeman (1982)

$\tau_c^* = 0.035(D_i/D_{50s})^{-0.94}$

D_i = Bed particle size for which i% is finer (mm). D_{84} used.

D_{50s} = Bed surface particle size for which 50% is finer (mm)

Rosgen (unpublished)

$\tau_c^* = 0.0384(D_i/D_{50})^{-0.887}$

D_i = Pavement D_{100} or bar sample D_{100} (mm)

D_{50} = Bed particle size for which 50% is finer

Allowable boundary shear stress can also be calculated from the following equation developed from the Shields diagram (Shields 1936)

$\tau_0 = \rho V^2/[5.75\log(12.27R/k_s)]^2$

ρ = Density of waster (slugs/ft^3)   R = Hydraulic radius (ft)

k_s = Grain roughness (usually taken as 3.5 D_{84} (ft))   V = Mean velocity (ft/s)

C.5.2 Results

Table C-6 presents the results of the analysis.

Table C-6. Comparison of critical shear stress and actual shear stress (lbs/ft^2) for selected reference reaches and proposed reaches.

Shear Stress Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Parker and Klingeman	0.63	0.82	1.57	0.77	0.97	0.67
Rosgen	0.50	1.30	0.84	0.80	0.80	1.00
Shields	0.73	1.22	1.25	0.70	0.77	0.73
Average of Methods (Q_{bf})	0.62	1.12	1.22	0.75	0.85	0.80

Table C-6 (Continued). Comparison of critical shear stress and actual shear stress (lbs/ft^2) for selected reference reaches and proposed reaches.

Shear Stress Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Actual Shear Stress (Q_{bf})	0.43	0.79	0.95	0.57	0.66	0.62
Actual Shear Stress (Q_2)	0.51	0.98	1.16	0.73	0.87	0.79
Actual Shear Stress (Q_{10})	0.70	1.35	1.57	0.91	1.07	0.96
Actual Shear Stress (Q_{25})	0.80	1.51	1.73	1.11	1.26	1.13
Actual Shear Stress (Q_{50})	0.87	1.61	1.84	1.37	1.40	1.32
Actual Shear Stress (Q_{100})	0.92	1.71	1.94	1.65	1.56	1.48
Actual Shear Stress (Q_{500})	1.04	1.88	2.14	2.46	1.59	2.02
Design Shear Stress (Q_{bf})	0.53-0.81	1.19	0.92	N/A	N/A	N/A

* Calculated critical shear stress values based on D_{84}.

C.5.3 Conclusions

Results yielded an acceptable range of critical shear stress values for bankfull discharge conditions. When compared with actual shear stress values, average critical shear stress values correspond closely to actual shear stress at Q_2 indicating that incipient motion is likely to occur at discharges slightly above bankfull. Assuming that the bankfull discharge is responsible for incipient motion, it could be concluded that the reference reaches are experiencing aggradation. As discussed earlier, significant aggradation was not observed in the reference reaches and variation among the results could indicate that dynamic equilibrium can occur within a range of shear stress values. In fact, the higher critical shear stress could be a result of bed armoring which was observed at all sites. The effects of bed armoring are discussed in more detail in C.7. Assuming similar bed composition for the proposed design reaches, design shear stress values at Q_{bf} lie within the range of values presented in Table C-4 for average critical shear stress.

C.6 SEDIMENT TRANSPORT ANALYSIS

Sediment transport analysis is generally the most challenging component of a hydraulic analysis due to the difficulty in collecting bedload data during high flows. Sediment transport data are essential for calibrating sediment transport models. Without such data, model calibration may not be possible and modeling results should be treated as suspect.

Despite these difficulties, understanding sediment transport and channel resistance to scour and deposition are essential for both understanding existing channel conditions and completing channel designs capable of conveying the available flow and sediment load while minimizing channel failure risk. As a design parameter, sediment transport analyses focus on providing for sediment continuity, a factor that is necessary for designing a stable channel. In this context, channel stability implies that although the channel configuration may change over time, the channel's cross-section dimensions remain consistent over time and space. Under the stable channel condition, rates of erosion and deposition are approximately balanced as the channel erodes its outer banks and stores sediment in bars.

C.6.1 Methods

In the absence of bed load data, sediment transport analyses relied on surface and sub surface sediment sampling from reference reaches. Sediment data and estimated transport rates from upstream and downstream reference reaches formed the boundary conditions for sediment input and output rates for the restoration project area. Due to the variability of results from available methods, objectives of the analysis focused on demonstrating sediment transport continuity rather than predicting actual transport rates. To demonstrate continuity, comparisons were drawn between transport rates for upstream reference reaches, proposed project reaches and a downstream reference reach. For this analysis, it is assumed that bed material within the project reaches will be substantially similar to that found in the respective reference reaches.

There are numerous equations that have been developed to predict sediment transport rates. Equations are generally derived from laboratory flume experiments that used sand bed channels to develop sediment transport relationships. Models based on field data collected in alluvial, gravel-bed rivers are less common. The following methods were selected for their applicability to gravel bed rivers.

<u>Meyer-Peter and Müller (1948) using D_{50ss}</u>

$q_b = 8[\rho_s/(\rho_s-\rho)]\ \sqrt{(g/\rho)}\ [(n'/n_t)^{3/2}\ \rho SD - 0.047(\rho_s-\rho)\ D_{50ss}]^{3/2}$

q_b = Bed load transport rate per unit width (kg/s)
ρ_s = Density of sediment (kg/m^3) — ρ = Density of water (kg/m^3)
g = gravitational acceleration (m/s^2) — S = Slope (m/m)
n_t = Roughness of bed = $S^{1/2}\ R^{2/3}/V$ — n' = particle roughness = $D_{90}{}^{1/6}/26$
R = Hydraulic radius (m) — V = Mean velocity (m/s)
D = Average flow depth (m)
D_{90} = Particle size for which 90% is finer (mm)
D_{50ss} = Subsurface Particle size for which 50% is finer (mm)

<u>Ackers and White (1973) using D_i</u>

$q_b = G_{gri}\ f_i\ \rho_s\ D_i\ V\ (V/u^*)^n \qquad 8f_i[\rho_s/(\rho_s-\rho)]\ \sqrt{(g/\rho)}\ [(n'/n_t)^{3/2}\ \rho SD - \tau^*_{ci}\ (\rho_s-\rho)\ D_{50ss}]^{3/2}$

q_b = Bed load transport rate per unit width (kg/s)
ρ_s = Density of sediment (kg/m^3)
G_{gri} = Dimensionless transport rate for ith class
f_i = Proportion of subsurface material in ith class (i used = 84)
D_i = Particle size for which ith % is finer (mm) (i used = 84)
V = Mean velocity (m/s) — D = Average flow depth (m)
u* = Shear velocity = $\sqrt{(gDS)}$ — S = Slope (m/m)

C.6.2 Results

Figures C-3 through C-8 present sediment transport rates for the proposed channels at various discharges. Each figure displays a plot of sediment transport rate in relation to valley stationing. The intent of this exercise is to evaluate sediment transport continuity between upstream reaches, project area reaches and downstream reaches. In general, sediment transport rate is expected to increase in the downstream direction as discharge and sediment load increase.

Station -5000 represents the Bandmann Flats reference reach in the CFR below the confluence. Station 5000 represents the design CFR channel below the confluence. Upstream of station 5000, the BFR and CFR are displayed separately at stations 7500 and 15,000 before entering the confluence. At station 7500, the sum of the transport rates for the CFR and BFR approximates the required transport rate for the CFR below the confluence at station 5000. Station 18000 represents an upstream reference reach in CFR3B.

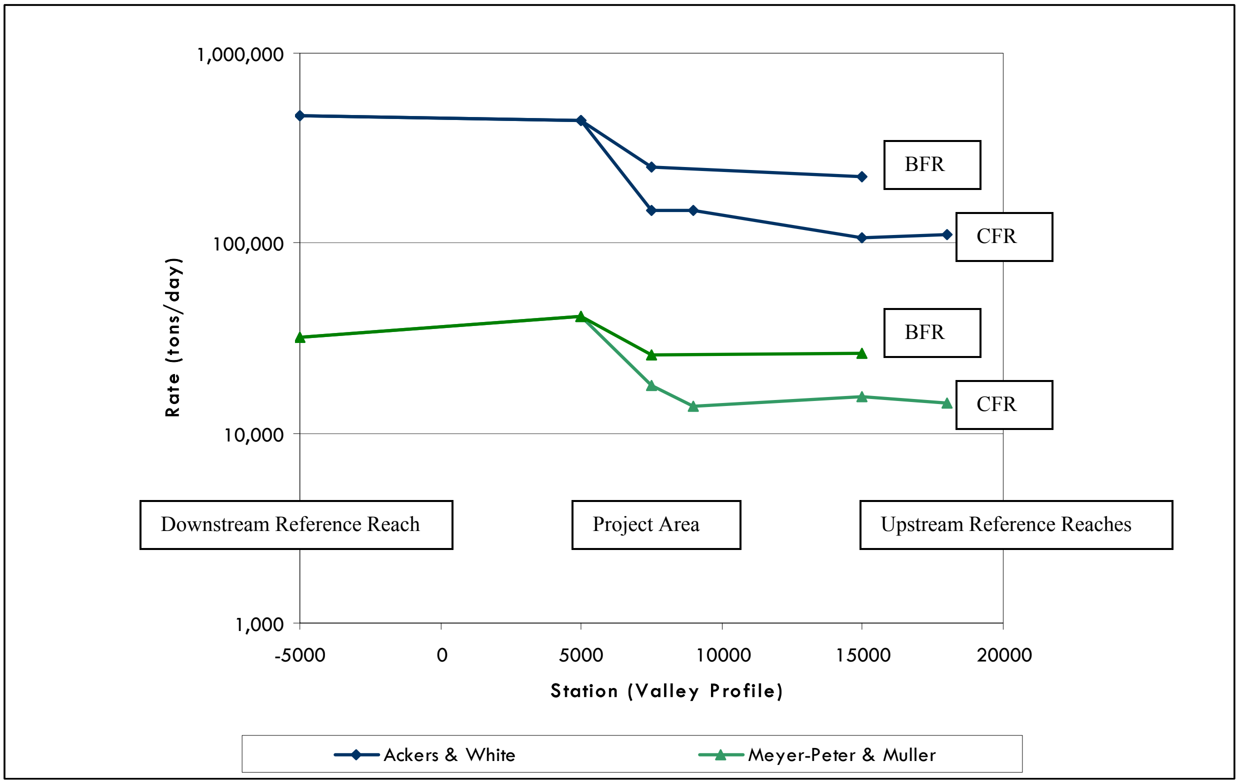


Figure C-3. Sediment Transport Rates for Proposed Channels at Bankfull Discharge.

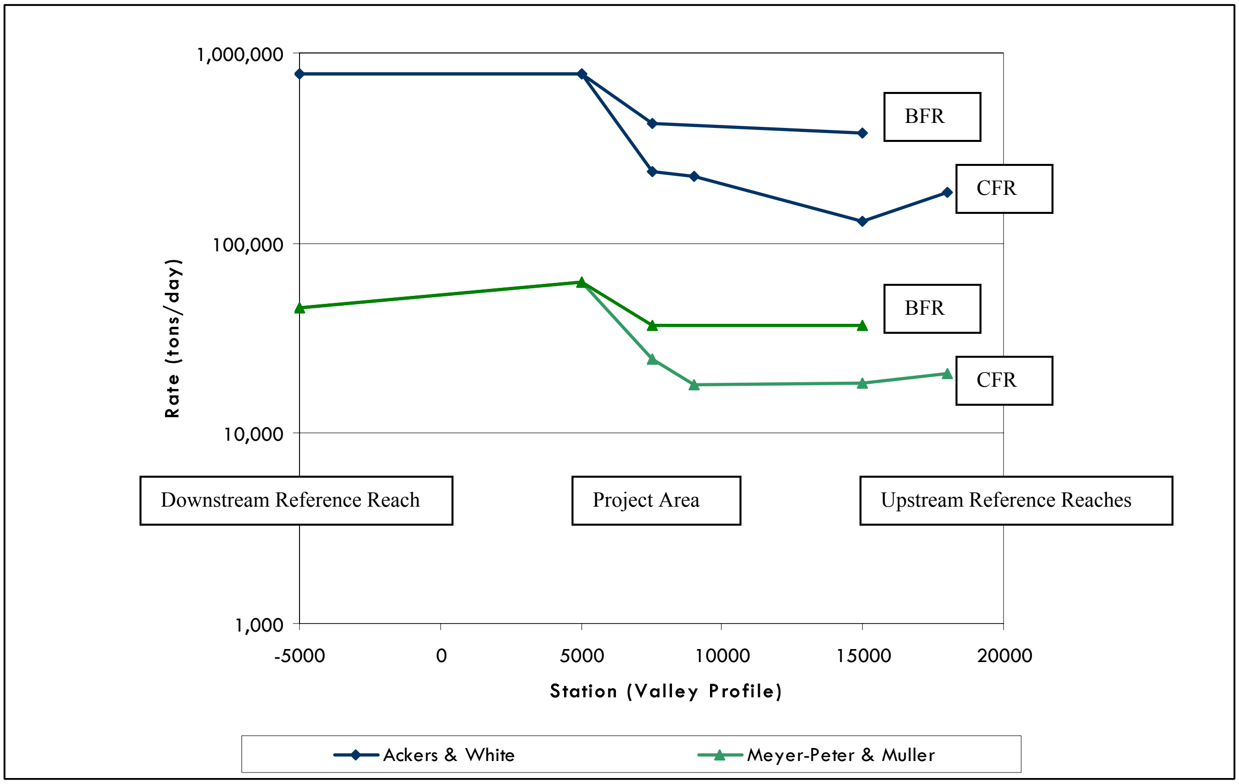


Figure C-4. Sediment Transport Rates for Proposed Channels at Q_2 Discharge.

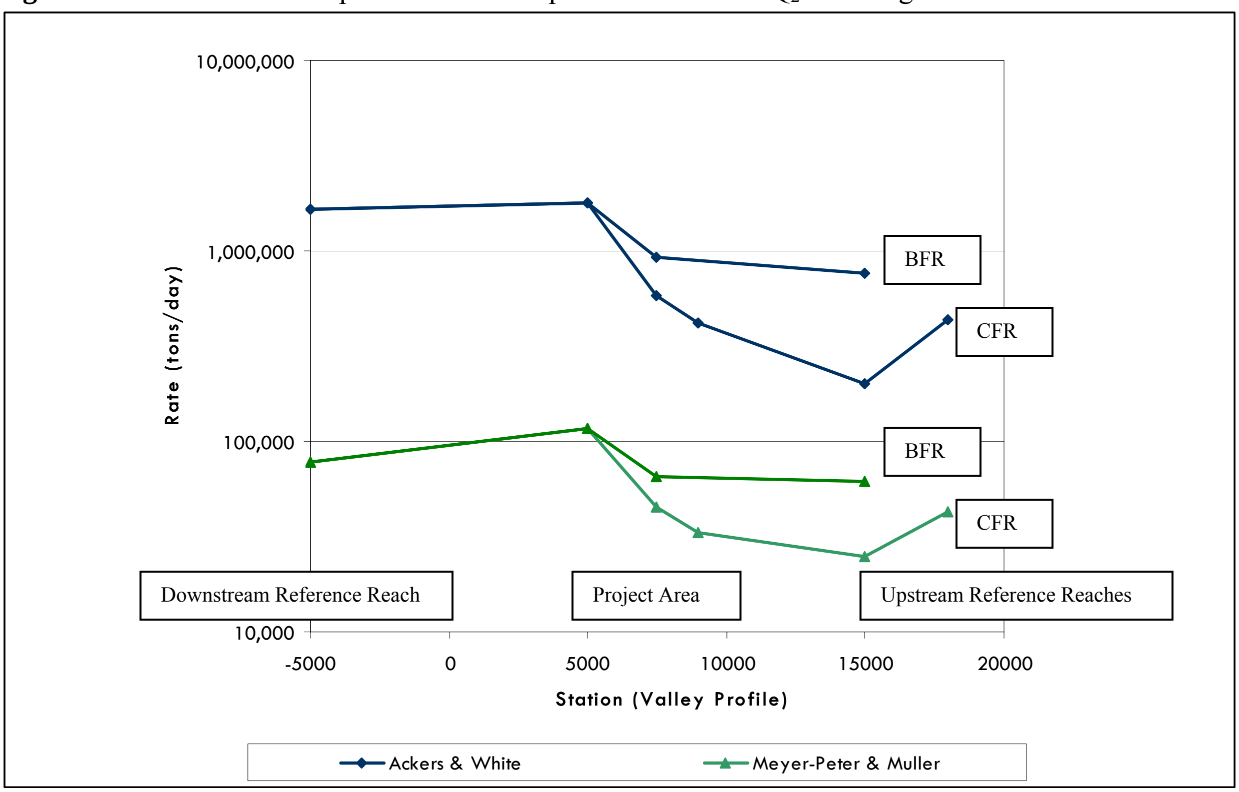


Figure C-5. Sediment Transport Rates for Proposed Channels at Q_{10} Discharge.

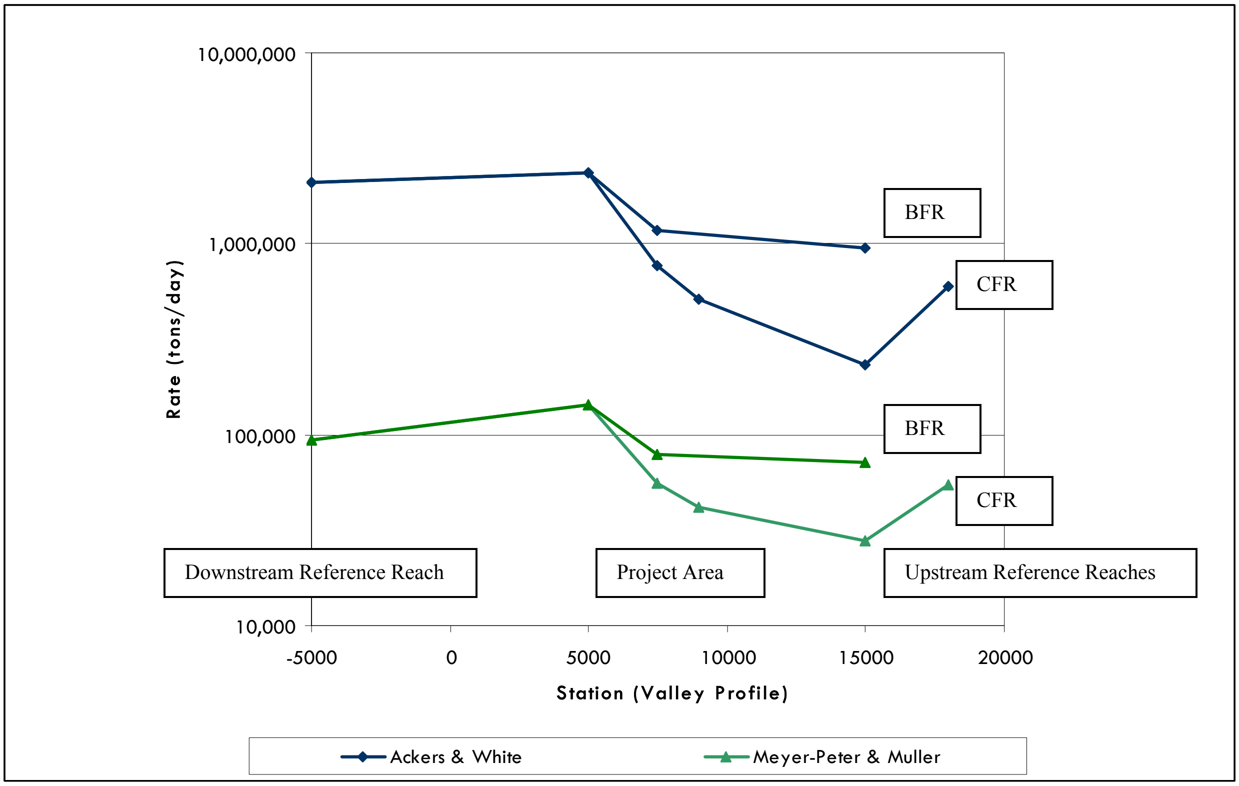


Figure C-6. Sediment Transport Rates for Proposed Channels at Q_{25} Discharge.

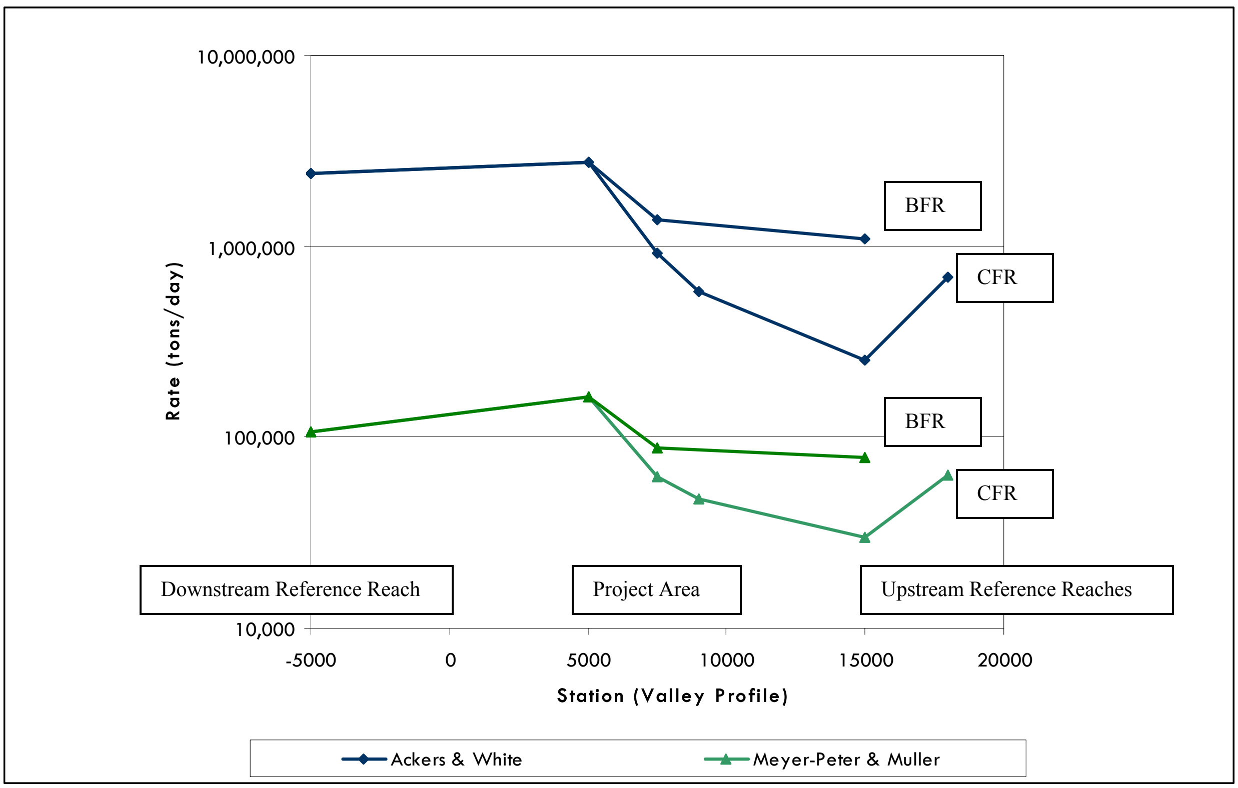


Figure C-7. Sediment Transport Rates for Proposed Channels at Q_{50} Discharge.

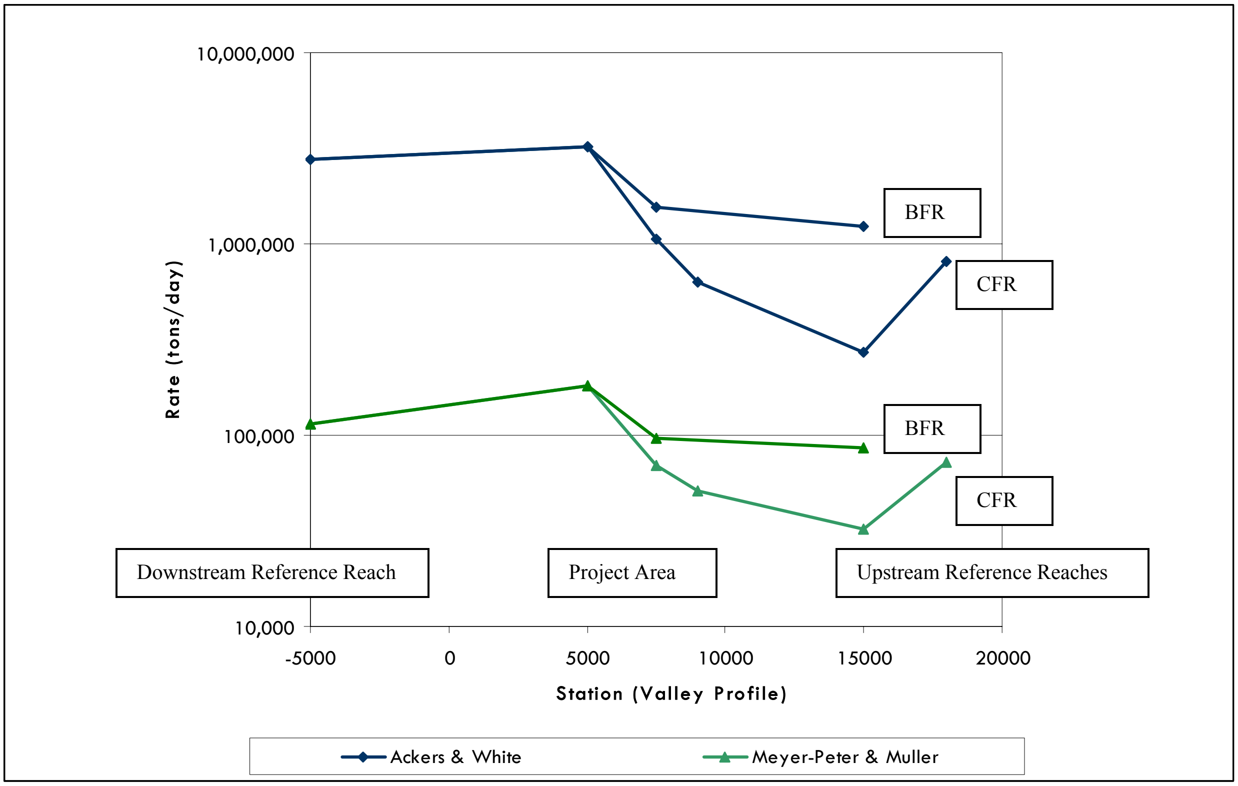


Figure C-8. Sediment Transport Rates for Proposed Channels at Q_{100} Discharge.

C.6.3 Conclusions

As shown in figures C-3 through C-8 results produced a wide range of values for sediment transport rates. As indicated previously, this is common with sediment transport analyses. Results were found to be sensitive to slight changes in input variables. Given this sensitivity and unacceptable range of values, conclusions focus on degree of sediment continuity between upstream and downstream reaches.

Results indicate that sediment transport capability increases in the downstream direction, implying that conveyance is adequate and reaches will not experience aggradation. When compared with the upstream reference reach, the proposed CFR3/2 reach appears to be unable to transport the potential sediment input from the upstream CFR3 reference at higher discharges. Although the difference in transport rates may not be significant, it could be attributed to the floodplain constriction that occurs as the CFR3 reach transitions to a more confined valley type in CFR2. More detailed modeling and further review of this transition will occur in the final design phase. Similarly, the potential sediment output from the proposed CFR1 reach appears to be greater than the transport capacity of the downstream CFR1 Bandmann reference reach at higher discharges. This could be due in part to the location of the CFR1 Bandmann reference below the dam. It is likely that the dam has altered the substrate composition and effects of peak discharges on this reach. Again, the difference may be insignificant but it warrants further review of potential effects of downstream deposition.

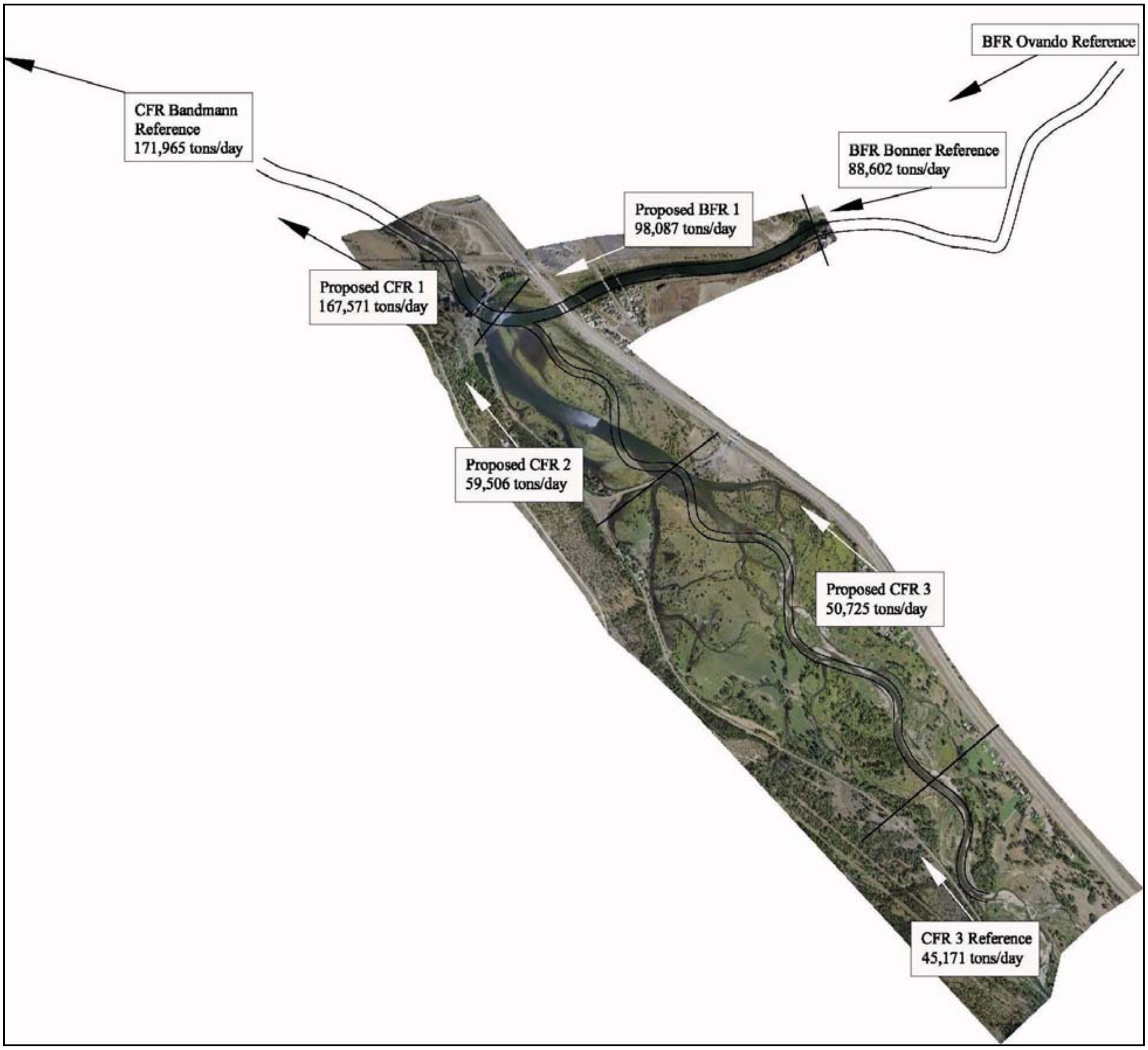


Figure C-9. Illustration of sediment transport continuity for selected reference reaches and project reaches at bankfull discharge. Values shown are averages values for all methods

In summary, a range of 3 to 30 percent difference in average sediment transport capability was observed. Again, this range is attributed to the sensitivity of the methods to slight changes in input variables and transitions in valley morphology. Except for existing conditions, percent difference was observed to increase as discharge increased. Results for Q_{bf} and Q_2 produced the lowest range of differences. Given the sensitivity of the methods and range of differences for existing conditions (21 to 23 percent), the analysis appears to provide an adequate demonstration of sediment transport continuity between upstream, restoration project area and downstream reaches. However, further review of effects of sediment transport discontinuity should be evaluated during final design.

C.7 SCOUR ANALYSIS

Several types of scour occur in rivers. General scour is a result of unbalanced sediment transport, which may include bed degradation over long periods of time or during extreme flow events. Contraction scour occurs when flow accelerates through an abrupt channel or floodplain constriction. Local scour occurs in the vicinity of an object that causes turbulent flow such as a drop structure, bridge pier, bridge abutment, isolated boulder or fallen tree. Bend scour occurs on the outside of channel meanders and is caused by turbulent flow generated by changes in flow vectors.

Scour can occur under live-bed or clear-water conditions. Live-bed conditions exist when the sediment transport rate is at or near the sediment transport capacity. Under live-bed conditions, erosion is offset by deposition. Clear-water conditions exist when the sediment transport rate is significantly less than sediment transport capacity. In such a case, potential exists for erosion without deposition. Accordingly, clear-water conditions typically produce greater scour depths than live-bed conditions.

Channel armoring can also affect scour. Armoring is common in gravel bed rivers due to the wide range of particle sizes available for transport. Armoring occurs when shear stress is insufficient to mobilize a portion of the bed material. Under these conditions, smaller particles are entrained and removed from the bed leaving the larger material behind in an armor layer. When the armor layer is disturbed as with channel construction, potential exists for scour in the reconstructed channel until an armor layer re-forms.

C.7.1 Methods

Most scour analysis methods are empirically derived from experiments or field measurements and can be biased toward site-specific experiment conditions. Moreover, most methods have been calibrated in sand bed channels. Usually, these methods grossly overestimate scour in gravel-bed rivers. Efforts focused on using equations that are applicable to gravel-bed rivers. In addition, a field-based scour analysis was implemented using scour chains. At the time of this report, no scour was recorded due to low peak discharge conditions.

A scour analysis was completed for the proposed project reaches to evaluate general scour and bend scour. General scour was analyzed to determine the required scour depth to re-form the armor layer in the new channel. Bend scour was analyzed to validate maximum pool depths.
The following equation was used to predict the depth of degradation that would need to occur to re-form an armor layer.

<u>U.S. Bureau of Reclamation (1984)</u>
$Y_s = Y_a(1/P_c-1)$
Y_s = Depth of scour required to form armor layer (ft)
Y_a = Thickness of armor layer (ft). Assumed to be twice critical particle size.
P_c = Fraction of material coarser than the critical particle size

The following equations were used to predict the depth of bend scour in the design meanders.

<u>Thorne (1997)</u>
$Y_{max} = Y_u[2.07\text{-}0.19\log(R_c/W_u\text{-}2)]$
Y_{max} = Maximum water depth in bend (ft)
Y_u = Maximum water depth in crossing upstream of bend (ft)
R_c = Centerline radius of bend (ft)
W_u = Water surface width at upstream of bend (ft)

<u>Maynord (1996)</u>
$Y_{max} = Y_u[1.8\text{-}0.051(R_c/W_u) + 0.0084(W_u/ Y_u)]$

<u>U.S. Army Corps of Engineers (1994)</u>

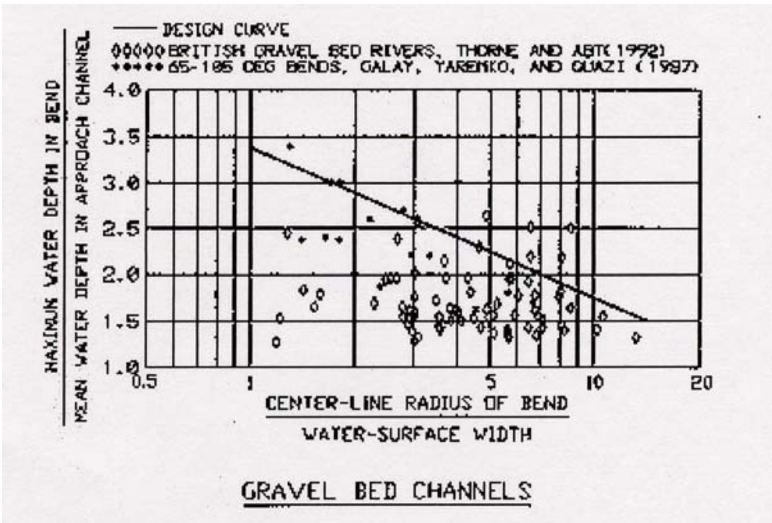


Figure C.10. U.S. Army Corps of Engineers momograph for predicting bend scour.

Table C-7 summarizes the results of the analysis.

Table C-7. Summary of predicted scour depths (ft) for selected reference reaches and proposed reaches at bankfull discharge.

Shear Stress Method	CFR3 Middle C4	CFR1 Bandmann F3/1	BFR1 Bonner B3	BFR Ovando C4	BFR Ovando B3	BFR Ovando F4
Depth of Scour to Form Armor	3.0	2.8	1.3	2.5	1.1	2.3
Depth of Scour to Form Armor Design Reaches	2.2	1.6	1.3			
Bend Scour - Thorne	5.9 - 7.9	11.2 - 13.2	7.7 - 8.8	5.9- 8.7	5.3 - 6.6	5.2 - 7.3
Bend Scour - Maynord	6.3 - 7.4	10.3 - 11.2	7.7 -8.3	6.2- 7.2	5.5 - 6.0	5.7 - 7.3
Bend Scour - USACE	7.6 - 9.9	11.9 -13.5	8.8 -9.9	7.9-11.9	5.2 - 6.3	6.1 - 9.3
Range of Values	5.9 - 9.9	10.3 - 13.5	7.7 -9.9	5.9-11.9	5.2 - 6.6	5.2 - 9.3
Avg Bend Scour	7.5	11.5	8.6	7.4	5.6	6.6
Observed Pool Max Depth	7.5	23.8	N/A	7.8	7.9	6.9
Selected Design	8.0 - 11.1	18.2 - 24.3	9.8 - 14.7	N/A	N/A	N/A

C.7.2 Results

Results indicate that a significant amount of scour would occur before the armor layer re-forms in the design channels. Grade control structures will be required to prevent the reconstructed channel bed from significant degradation. Frequency of grade control structures will depend on channel slope and threshold for allowable bed degradation. This is discussed in more detail in 4.5.

Bend scour results yielded an acceptable range of values for bankfull discharge. Except for the CFR1 Bandmann reach, average results corresponded closely with observed maximum pool depths in the reference reaches. The observed maximum pool depth in the CFR1 Bandmann reach occurred on the outside of a bend near a bedrock outcrop. A similar bedrock outcrop is present in the proposed CFR1 design reach near the dam. For this reason, the observed value was given greater emphasis when selecting design values. A supplemental check of 100-year scour depths was performed for the CFR1 and BFR1 proposed reaches, yielding values of 24.1 ft and 20.3 ft, respectively. Further consideration will be given to 100-yr scour depths and associated design implications in Phase III final design.

C.8 ICE ASSESSMENT

In Montana, where rivers can develop thick ice covers during the winter, ice jams can contribute significantly to channel scour and flooding. Although discharges may be relatively low, the stages of ice jam flooding may be among the highest on record. Ice jams typically occur repeatedly in the same locations and ice jam flooding tends to be localized.

In the U.S. Army Cold Regions Research and Engineering Laboratory Technical Note entitled "Methodology for Ice Jam Analysis" (1980), ice jams were classified as freezeup, breakup, floating and grounded jams. Freezeup jams are associated with the formation of frazil ice, which eventually forms a continuous ice cover. Breakup jams occur when ice cover is followed by a rapid increase in runoff and an ice dam forms. Floating jams are not anchored to the channel bed and flow passes beneath the jam. Grounded jams are anchored to the channel bed and flow passes over the jam and onto the floodplain.

Ice jams occur at locations where abrupt changes in ice conveyance occur. Common features that contribute to ice jams include transitions from steep to mild slope, confluences, tight meanders and channel obstructions. Nearly all of these features are present in the vicinity of the restoration project area. The history of ice jams occurrences on the BFR and CFR confirms this. Table C.9 presents a summary of historical ice jams in the vicinity of the restoration project area.

Table C-8. Summary of historical ice jams on BFR and CFR near Milltown Dam.*

River	Date	Location	Type	Size	Ice Stage	Discharge Stage
BFR	02-09-96	¾-mile upstream from Bonner	Breakup	N/A	16 ft Max. on record	5.2 ft
BFR	02-11-96	Milltown	Breakup	8'H 40' W 4 mi L	N/A	4.3 ft
CFR	01-08-30	Missoula	N/A	N/A		2.9 ft
CFR	01-11-77	Missoula	N/A	N/A	4.41 ft Max. annual	3.0 ft
CFR	11-28-93	Bonner	N/A	N/A	7.26 ft Max. annual	2.4 ft
CFR	10-10-95	Bonner	N/A	N/A	10.24 ft Max. annual	3.0 ft
CFR	02-08-96	Turah	Breakup	N/A	9.05 ft Max on record	5.6 ft

* U.S. Army Research and Development Center Cold Regions Research and Engineering Laboratory Ice Jam Database

Historically, ice jams have formed upstream of Milltown Dam at Stimson Dam and Turah Bridge. Upon removal of Stimson Dam, the potential increases for ice jams to move or form further downstream. On the BFR, potential areas for ice jam formation include obstructions from bridge piers and abutments on the Highway 200 bridge, Burlington Northern Railroad bridge and Interstate 90 bridges. In addition, the confluence could be another potential area for ice jam formation. On the CFR, ice jams are likely to reoccur at the Turah bridge. However, ice jams that move past the bridge could become lodged in CFR meander bends or steep-to-mild transition areas between riffles and pools. The confluence also presents another possible ice jam location on the CFR. Records indicate that ice jams on the CFR and BFR account for the maximum stage on record.

EPA has commissioned the U.S. Army Corps of Engineers to perform an ice analysis for pre and post dam removal conditions. The results of this analysis are not yet available. During Phase III final design, RDG and WWC will rely on the results of this analysis for potential post dam removal effects on the stability of the proposed design.

C.9 HEC-RAS OUTPUT

Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top W Chnl (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandmann	660	QBankfull	10400	8.57	0.84	1455.45	0.032	238.21	6.01	7.24	6.06
Bandmann	200	QBankfull	10400	9.29	0.81	1645.71	0.037	228.51	7.15	6.36	5.18
Bandmann	0	QBankfull	10400	9.21	0.72	1311.25	0.027	217.18	6.04	7.93	5.68
Blackfoot Gage	2228	QBankfull	6200	6.49	1.00	948.33	0.038	194.90	4.86	6.44	6.45
Blackfoot Gage	1999	QBankfull	6200	7.93	0.86	999.27	0.038	176.10	5.66	6.12	5.25
Blackfoot Gage	1829	QBankfull	6200	7.41	1.28	962.51	0.044	188.70	5.10	6.34	8.11
Blackfoot Gage	402	QBankfull	6200	10.01	0.69	1130.37	0.038	195.92	5.66	5.48	3.80
Blackfoot Gage	0	QBankfull	6200	6.96	0.90	1000.86	0.038	200.55	4.97	6.12	5.48
Ovando B	550	QBankfull	3500	4.77	0.81	638.49	0.038	162.48	3.87	5.56	4.49
Ovando B	0	QBankfull	3500	4.14	0.51	725.28	0.034	217.31	3.33	4.83	2.47
Ovando C	6200	QBankfull	3500	6.97	0.44	712.58	0.032	165.00	4.26	4.97	2.20
Ovando C	5340	QBankfull	3500	4.76	0.37	720.64	0.028	187.03	3.66	5.09	1.91
Ovando C	250	QBankfull	3500	4.93	0.90	575.70	0.036	167.14	3.44	6.08	5.46
Ovando C	0	QBankfull	3500	5.31	0.56	595.42	0.030	154.31	3.85	5.88	3.32
Ovando F	1490	QBankfull	3500	5.73	0.63	609.55	0.032	171.21	3.56	5.74	3.60
Ovando F	0	QBankfull	3500	5.33	0.60	720.70	0.038	174.75	4.12	4.86	2.92
Ref CFR 3	1730	QBankfull	3200	4.57	0.28	696.32	0.025	191.94	3.37	4.90	1.39
Ref CFR 3	1560	QBankfull	3200	4.26	0.45	660.76	0.030	192.62	3.24	5.10	2.28
Ref CFR 3	0	QBankfull	3200	5.14	0.57	502.04	0.027	142.28	3.48	6.46	3.67
CFR 3/CFR 2	14000	QBankfull	3200	4.57	0.5	545.28	0.028	147.88	3.69	5.87	2.91
CFR 3/CFR 2	8000	QBankfull	3200	4.48	0.77	532.28	0.034	147.53	3.61	6.01	4.65
CFR 3/CFR 2	6000	QBankfull	3200	4.62	0.81	535.67	0.035	146.00	3.65	6.00	4.89
CFR 1	1000	QBankfull	10400	7.70	1.18	1502.56	0.040	243.00	6.18	6.92	8.20
CFR 1	700	QBankfull	10400	7.70	1.19	1502.36	0.040	243.00	6.18	6.92	8.21
CFR 1	400	QBankfull	10400	7.69	1.19	1501.95	0.040	243.00	6.18	6.92	8.21
BFR 1	1000	QBankfull	6200	6.17	0.92	974.06	0.037	196.00	4.97	6.37	5.87
BFR 1	700	QBankfull	6200	6.17	0.92	974.27	0.037	196.00	4.97	6.36	5.87
BFR 1	400	QBankfull	6200	6.17	0.92	974.14	0.037	196.00	4.97	6.36	5.87

Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandmann	660	Q2	14900	10.14	0.99	1876.35	0.032	277.17	7.57	8.19	8.12
Bandmann	200	Q2	14900	10.85	1.05	2020.97	0.037	243.30	8.71	7.47	7.84
Bandmann	0	Q2	14900	10.66	0.89	1633.51	0.027	225.18	7.49	9.15	8.13
Blackfoot Gage	2228	Q2	8700	7.77	1.17	1217.41	0.038	220.32	6.14	7.25	8.52
Blackfoot Gage	1999	Q2	8700	9.20	1.09	1235.81	0.038	191.69	6.93	7.11	7.73
Blackfoot Gage	1829	Q2	8700	8.71	1.53	1216.97	0.044	201.36	6.24	7.17	10.97
Blackfoot Gage	402	Q2	8700	11.23	0.88	1404.76	0.038	228.21	6.85	6.39	5.64
Blackfoot Gage	0	Q2	8700	8.13	1.11	1254.19	0.038	221.77	6.13	7.04	7.79
Ovando B	550	Q2	5400	5.79	1.08	888.38	0.038	290.11	4.81	6.68	7.23
Ovando B	0	Q2	5400	5.14	0.66	944.95	0.034	223.69	4.33	5.74	3.80
Ovando C	6200	Q2	5400	8.04	0.62	906.63	0.032	186.59	5.33	6.11	3.80
Ovando C	5340	Q2	5400	5.75	0.49	1008.88	0.028	294.84	4.62	6.06	2.96
Ovando C	250	Q2	5400	6.07	1.09	776.06	0.036	185.84	4.59	7.03	7.68
Ovando C	0	Q2	5400	6.47	0.72	789.54	0.030	180.21	4.96	6.95	5.03
Ovando F	1490	Q2	5400	6.85	0.79	808.33	0.032	183.01	4.65	6.73	5.30
Ovando F	0	Q2	5400	6.56	0.78	938.99	0.038	180.80	5.35	5.78	4.51
Ref CFR 3	1730	Q2	4600	5.42	0.34	988.22	0.025	402.85	4.22	5.53	1.85
Ref CFR 3	1560	Q2	4600	5.15	0.50	930.83	0.030	316.01	4.14	5.61	2.80
Ref CFR 3	0	Q2	4600	5.97	0.70	653.13	0.027	201.31	4.31	7.44	5.24
CFR 3/CFR 2	14000	Q2	4600	5.06	0.55	1932.04	0.028	3001.84	4.17	6.29	3.44
CFR 3/CFR 2	8000	Q2	4600	5.25	0.91	941.46	0.034	612.93	4.36	6.74	6.14
CFR 3/CFR 2	6000	Q2	4600	5.43	0.99	785.27	0.035	316.54	4.46	6.86	6.81
CFR 1	1000	Q2	14900	9.18	1.47	1873.83	0.040	255.67	7.67	7.99	11.74
CFR 1	700	Q2	14900	9.18	1.47	1873.80	0.040	255.67	7.67	7.99	11.74
CFR 1	400	Q2	14900	9.18	1.47	1873.72	0.040	255.66	7.67	7.99	11.74
BFR 1	1000	Q2	8670	7.28	1.13	1194.93	0.037	203.06	6.08	7.28	8.21
BFR 1	700	Q2	8670	7.28	1.13	1195.06	0.037	203.05	6.08	7.28	8.20
BFR 1	400	Q2	8670	7.27	1.13	1194.73	0.037	203.04	6.07	7.28	8.21

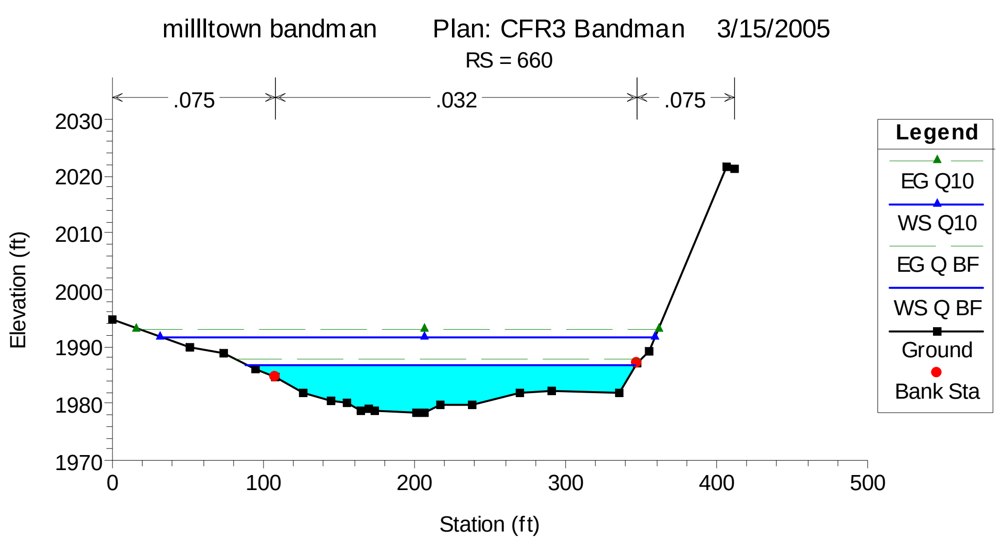
Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandman	660	Q10	25900	13.31	1.29	2840.89	0.032	326.81	10.74	9.90	12.77
Bandman	200	Q10	25900	13.97	1.54	2800.56	0.037	256.48	11.83	9.51	14.65
Bandman	0	Q10	25900	13.58	1.23	2354.18	0.027	254.91	10.41	11.40	14.05
Blackfoot Gage	2228	Q10	14600	10.16	1.50	1778.33	0.038	244.36	8.54	8.65	12.97
Blackfoot Gage	1999	Q10	14600	11.49	1.55	1699.95	0.038	214.84	9.22	8.92	13.84
Blackfoot Gage	1829	Q10	14600	11.04	2.03	1721.56	0.044	225.14	8.57	8.71	17.68
Blackfoot Gage	402	Q10	14600	13.50	1.25	1931.93	0.038	236.57	9.12	7.96	9.93
Blackfoot Gage	0	Q10	14600	10.32	1.50	1761.29	0.038	240.58	8.33	8.63	12.96
Ovando B	550	Q10	7800	6.85	1.32	1235.20	0.038	351.09	5.87	7.63	10.08
Ovando B	0	Q10	7800	6.20	0.82	1191.89	0.034	242.76	5.39	6.65	5.48
Ovando C	6200	Q10	7800	9.13	0.84	1125.11	0.032	222.86	6.42	7.30	6.10
Ovando C	5340	Q10	7800	6.82	0.60	1325.05	0.028	294.84	5.69	6.95	4.17
Ovando C	250	Q10	7800	7.23	1.33	1006.86	0.036	207.28	5.75	8.06	10.73
Ovando C	0	Q10	7800	7.65	0.88	1193.14	0.030	563.14	6.06	7.94	7.02
Ovando F	1490	Q10	7800	8.06	0.95	1039.40	0.032	203.14	5.87	7.69	7.31
Ovando F	0	Q10	7800	7.87	0.97	1179.04	0.038	184.48	6.66	6.69	6.51
Ref CFR 3	1730	Q10	9700	7.88	0.42	2113.13	0.025	471.12	6.68	6.71	2.84
Ref CFR 3	1560	Q10	9700	7.69	0.61	2012.35	0.030	457.90	6.67	6.71	4.09
Ref CFR 3	0	Q10	9700	8.23	1.07	1211.37	0.027	274.53	6.57	9.86	10.58
CFR 3/CFR 2	14000	Q10	9700	5.97	0.65	4658.07	0.028	3005.47	5.08	7.11	4.65
CFR 3/CFR 2	8000	Q10	9700	7.14	1.12	2141.19	0.034	650.90	6.26	7.95	8.94
CFR 3/CFR 2	6000	Q10	9700	7.54	1.46	1497.53	0.035	358.73	6.57	8.88	12.97
CFR 1	1000	Q10	25900	12.16	2.04	2671.39	0.040	279.51	10.65	9.95	20.31
CFR 1	700	Q10	25900	12.16	2.04	2671.07	0.040	279.50	10.65	9.95	20.31
CFR 1	400	Q10	25900	12.16	2.04	2670.68	0.040	279.49	10.65	9.95	20.32
BFR 1	1000	Q10	14600	9.49	1.54	1659.37	0.037	216.35	8.29	8.95	13.78
BFR 1	700	Q10	14600	9.49	1.54	1659.50	0.037	216.34	8.29	8.95	13.77
BFR 1	400	Q10	14600	9.49	1.54	1659.18	0.037	216.33	8.29	8.95	13.78

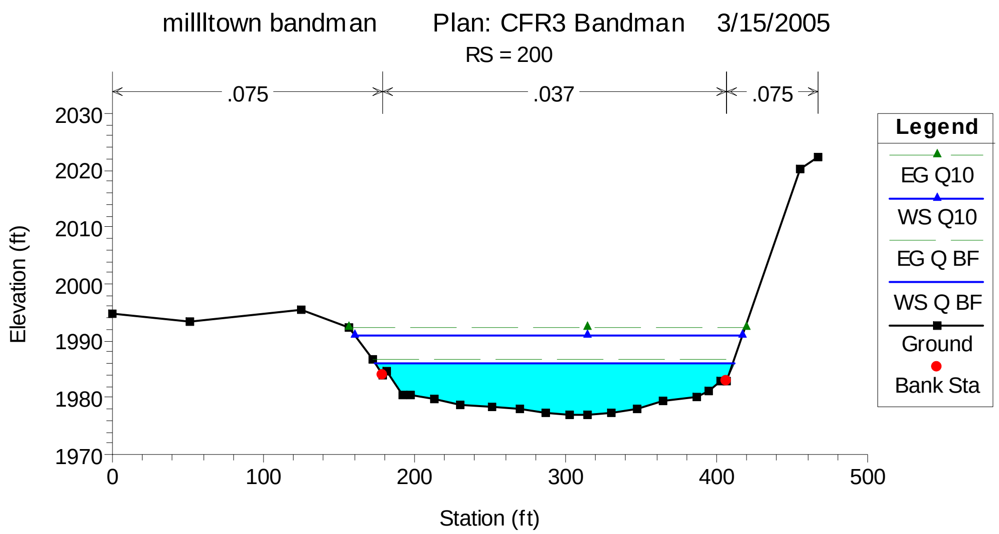
Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandman	660	Q25	31200	14.63	1.41	3283.44	0.032	343.38	12.06	10.54	14.83
Bandman	200	Q25	31200	15.26	1.75	3134.08	0.037	261.92	13.12	10.31	18.02
Bandman	0	Q25	31200	14.78	1.38	2662.00	0.027	256.65	11.61	12.27	16.90
Blackfoot Gage	2228	Q25	17300	11.12	1.62	2015.45	0.038	251.46	9.49	9.16	14.88
Blackfoot Gage	1999	Q25	17300	12.39	1.74	1897.33	0.038	223.87	10.12	9.59	16.68
Blackfoot Gage	1829	Q25	17300	11.95	2.23	1930.16	0.044	230.74	9.49	9.28	20.73
Blackfoot Gage	402	Q25	17300	14.40	1.39	2147.16	0.038	239.14	10.03	8.55	11.91
Blackfoot Gage	0	Q25	17300	11.20	1.66	1979.42	0.038	256.66	9.21	9.22	15.30
Ovando B	550	Q25	10500	7.88	1.53	1648.57	0.038	460.26	6.91	8.45	12.96
Ovando B	0	Q25	10500	7.23	0.98	1454.46	0.034	264.54	6.42	7.48	7.35
Ovando C	6200	Q25	10500	10.21	1.03	1366.82	0.032	224.50	7.50	8.33	8.58
Ovando C	5340	Q25	10500	7.99	0.68	1670.41	0.028	294.84	6.86	7.61	5.15
Ovando C	250	Q25	10500	8.08	1.72	1188.03	0.036	215.66	6.59	9.39	16.18
Ovando C	0	Q25	10500	8.61	1.01	1802.56	0.030	688.56	6.95	8.69	8.80
Ovando F	1490	Q25	10500	9.27	1.10	1297.24	0.032	217.41	7.07	8.53	9.38
Ovando F	0	Q25	10500	9.16	1.16	1418.22	0.038	185.71	7.95	7.53	8.74
Ref CFR 3	1730	Q25	12600	8.91	0.48	2600.22	0.025	474.82	7.71	7.32	3.52
Ref CFR 3	1560	Q25	12600	8.73	0.68	2490.26	0.030	457.90	7.72	7.26	4.95
Ref CFR 3	0	Q25	12600	9.25	1.24	1490.13	0.027	274.53	7.58	10.85	13.44
CFR 3/CFR 2	14000	Q25	12600	6.36	0.7	5837.69	0.028	3007.04	5.47	7.43	5.17
CFR 3/CFR 2	8000	Q25	12600	8.02	1.2	2720.48	0.034	668.46	7.14	8.40	10.08
CFR 3/CFR 2	6000	Q25	12600	8.48	1.67	1845.99	0.035	377.66	7.52	9.71	16.25
CFR 1	1000	Q25	31200	13.40	2.28	3022.84	0.040	289.40	11.89	10.70	24.37
CFR 1	700	Q25	31200	13.40	2.28	3022.73	0.040	289.39	11.89	10.70	24.37
CFR 1	400	Q25	31200	13.40	2.28	3022.55	0.040	289.39	11.89	10.70	24.37
BFR 1	1000	Q25	17300	10.37	1.70	1851.66	0.037	221.62	9.17	9.57	16.29
BFR 1	700	Q25	17300	10.37	1.70	1851.84	0.037	221.61	9.17	9.57	16.28
BFR 1	400	Q25	17300	10.37	1.70	1851.59	0.037	221.61	9.17	9.57	16.29

Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandman	660	Q50	35000	15.53	1.48	3596.83	0.032	354.64	12.96	10.95	16.22
Bandman	200	Q50	35000	16.13	1.89	3364.81	0.037	270.56	13.99	10.84	20.48
Bandman	0	Q50	35000	15.60	1.47	2872.46	0.027	261.79	12.43	12.83	18.91
Blackfoot Gage	2228	Q50	19200	11.76	1.71	2176.75	0.038	256.17	10.13	9.50	16.20
Blackfoot Gage	1999	Q50	19200	12.99	1.86	2032.84	0.038	229.55	10.72	10.02	18.67
Blackfoot Gage	1829	Q50	19200	12.56	2.37	2076.44	0.044	258.38	10.09	9.66	22.86
Blackfoot Gage	402	Q50	19200	14.99	1.49	2288.32	0.038	240.23	10.61	8.94	13.35
Blackfoot Gage	0	Q50	19200	11.78	1.76	2130.50	0.038	265.28	9.78	9.60	16.93
Ovando B	550	Q50	14000	9.09	1.64	2212.71	0.038	484.10	8.12	8.98	14.72
Ovando B	0	Q50	14000	8.41	1.16	1770.38	0.034	271.70	7.60	8.36	9.72
Ovando C	6200	Q50	14000	11.49	1.24	1655.54	0.032	226.42	8.78	9.37	11.61
Ovando C	5340	Q50	14000	9.42	0.75	2092.36	0.028	294.84	8.29	8.26	6.19
Ovando C	250	Q50	14000	8.81	2.36	1347.59	0.036	216.32	7.33	11.17	26.35
Ovando C	0	Q50	14000	9.59	1.14	2514.73	0.030	732.82	7.86	9.43	10.77
Ovando F	1490	Q50	14000	10.67	1.26	1603.98	0.032	219.08	8.48	9.40	11.80
Ovando F	0	Q50	14000	10.65	1.38	1695.85	0.038	187.12	9.44	8.44	11.62
Ref CFR 3	1730	Q50	14700	9.61	0.52	2929.56	0.025	474.82	8.40	7.70	3.98
Ref CFR 3	1560	Q50	14700	9.43	0.73	2812.56	0.030	457.90	8.42	7.61	5.53
Ref CFR 3	0	Q50	14700	9.92	1.35	1674.31	0.027	274.53	8.26	11.48	15.48
CFR 3/CFR 2	14000	Q50	14700	6.62	0.72	6629.18	0.028	3008.1	5.74	7.61	5.47
CFR 3/CFR 2	8000	Q50	14700	8.61	1.25	3116.12	0.034	680.19	7.72	8.67	10.83
CFR 3/CFR 2	6000	Q50	14700	9.11	1.81	2085.05	0.035	390.12	8.14	10.24	18.54
CFR 1	1000	Q50	35000	14.23	2.44	3265.56	0.040	296.03	12.72	11.19	27.26
CFR 1	700	Q50	35000	14.23	2.44	3265.53	0.040	296.03	12.72	11.19	27.26
CFR 1	400	Q50	35000	14.23	2.44	3265.49	0.040	296.03	12.72	11.19	27.26
Missoula Gage	610	Q50	35000	13.79	1.20	3217.25	0.028	273.93	12.48	11.19	13.45
Missoula Gage	0	Q50	35000	10.48	1.86	2658.83	0.028	285.99	9.52	13.27	24.66
BFR 1	1000	Q50	19200	10.95	1.81	1981.94	0.037	225.12	9.75	9.98	18.05
BFR 1	700	Q50	19200	10.95	1.81	1982.16	0.037	225.11	9.75	9.97	18.04
BFR 1	400	Q50	19200	10.95	1.81	1981.98	0.037	225.11	9.75	9.98	18.05

Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandman	660	Q100	38600	16.37	1.54	3898.67	0.032	363.92	13.80	11.28	17.38
Bandman	200	Q100	38600	16.91	2.02	3585.42	0.037	302.49	14.77	11.30	22.79
Bandman	0	Q100	38600	16.34	1.56	3070.74	0.027	274.29	13.17	13.33	20.81
Blackfoot Gage	2228	Q100	21000	12.34	1.78	2327.34	0.038	260.49	10.71	9.79	17.41
Blackfoot Gage	1999	Q100	21000	13.54	1.97	2161.92	0.038	239.96	11.27	10.39	20.48
Blackfoot Gage	1829	Q100	21000	13.11	2.50	2228.80	0.044	284.41	10.65	10.01	25.03
Blackfoot Gage	402	Q100	21000	15.51	1.59	2413.79	0.038	241.19	11.14	9.30	14.80
Blackfoot Gage	0	Q100	21000	12.29	1.86	2267.41	0.038	266.82	10.30	9.94	18.45
Ovando B	550	Q100	17500	10.20	1.78	2834.42	0.038	682.01	9.23	9.54	16.95
Ovando B	0	Q100	17500	9.46	1.33	2055.61	0.034	271.70	8.65	9.12	12.08
Ovando C	6200	Q100	17500	12.69	1.41	1929.64	0.032	228.22	9.99	10.20	14.36
Ovando C	5340	Q100	17500	10.78	0.80	2492.74	0.028	294.84	9.65	8.78	7.05
Ovando C	250	Q100	17500	9.29	3.15	1451.25	0.036	216.74	7.81	13.05	41.14
Ovando C	0	Q100	17500	10.42	1.25	3121.01	0.030	734.30	8.61	10.00	12.50
Ovando F	1490	Q100	17500	11.95	1.39	1884.93	0.032	220.59	9.76	10.13	14.11
Ovando F	0	Q100	17500	12.00	1.57	1948.41	0.038	188.40	10.78	9.22	14.51
Ref CFR 3	1730	Q100	16900	10.31	0.55	3265.96	0.025	474.82	9.11	8.04	4.41
Ref CFR 3	1560	Q100	16900	10.15	0.76	3141.79	0.030	457.90	9.14	7.92	6.05
Ref CFR 3	0	Q100	16900	10.58	1.46	1855.67	0.027	274.53	8.92	12.08	17.59
CFR 3/CFR 2	14000	Q100	16900	6.88	0.74	7411.3	0.028	3009.14	6.00	7.77	5.75
CFR 3/CFR 2	8000	Q100	16900	9.19	1.29	3515.38	0.034	691.83	8.30	8.93	11.55
CFR 3/CFR 2	6000	Q100	16900	9.71	1.95	2325.35	0.035	402.25	8.74	10.74	20.91
CFR 1	1000	Q100	38600	14.98	2.58	3490.31	0.040	302.04	13.47	11.62	29.97
CFR 1	700	Q100	38600	14.98	2.58	3490.54	0.040	302.05	13.47	11.62	29.96
CFR 1	400	Q100	38600	14.98	2.58	3490.85	0.040	302.06	13.47	11.62	29.95
BFR 1	1000	Q100	21000	11.48	1.91	2101.77	0.037	227.53	10.28	10.33	19.71
BFR 1	700	Q100	21000	11.48	1.91	2101.98	0.037	227.53	10.28	10.33	19.70
BFR 1	400	Q100	21000	11.48	1.91	2101.80	0.037	227.52	10.28	10.33	19.71

Reach	River Station	Profile	Q Total (cfs)	Max Chl Dpth (ft)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Mann Wtd Chnl	Top Wtd (ft)	Hydr Depth C (ft)	Vel Chnl (ft/s)	Power Chan (lb/ft s)
Bandman	660	Q500	46800	18.19	1.65	4565.26	0.032	366.85	15.62	11.92	19.67
Bandman	200	Q500	46800	18.66	2.25	4230.19	0.037	423.44	16.52	12.16	27.40
Bandman	0	Q500	46800	17.93	1.75	3532.47	0.027	311.43	14.76	14.38	25.15
Blackfoot Gage	2228	Q500	24900	13.49	1.94	2632.06	0.038	269.03	11.86	10.40	20.22
Blackfoot Gage	1999	Q500	24900	14.61	2.21	2426.92	0.038	252.68	12.34	11.17	24.72
Blackfoot Gage	1829	Q500	24900	14.24	2.72	2549.07	0.044	285.76	11.77	10.61	28.84
Blackfoot Gage	402	Q500	24900	16.58	1.79	2670.98	0.038	243.23	12.20	10.03	17.99
Blackfoot Gage	0	Q500	24900	13.35	2.05	2551.08	0.038	269.98	11.35	10.61	21.71
Ovando B	550	Q500	31000	14.38	1.32	6927.39	0.038	1017.83	13.41	8.75	11.54
Ovando B	0	Q500	31000	12.88	1.85	2986.30	0.034	271.70	12.07	11.39	21.06
Ovando C	6200	Q500	31000	16.51	2.00	2828.67	0.032	280.72	13.80	12.83	25.61
Ovando C	5340	Q500	31000	14.90	1.06	3707.74	0.028	294.84	13.77	10.69	11.30
Ovando C	250	Q500	31000	11.62	5.19	1978.37	0.036	233.36	10.14	17.49	90.74
Ovando C	0	Q500	31000	12.94	1.60	5048.90	0.030	787.73	11.01	11.78	18.81
Ovando F	1490	Q500	31000	16.20	1.82	2833.95	0.032	226.32	14.01	12.29	22.31
Ovando F	0	Q500	31000	16.37	2.21	2782.65	0.038	192.56	15.16	11.57	25.62
Ref CFR 3	1730	Q500	22100	11.93	0.61	4031.29	0.025	474.82	10.72	8.68	5.26
Ref CFR 3	1560	Q500	22100	11.79	0.83	3891.24	0.030	457.90	10.78	8.50	7.09
Ref CFR 3	0	Q500	22100	12.00	1.69	2247.50	0.027	274.53	10.34	13.35	22.58
CFR 3/CFR 2	14000	Q500	22100	7.46	0.77	9152.91	0.028	3010	6.58	8.08	6.25
CFR 3/CFR 2	8000	Q500	22100	10.46	1.39	4411.62	0.034	717.28	9.58	9.47	13.12
CFR 3/CFR 2	6000	Q500	22100	11.01	2.24	2863.84	0.035	428.18	10.04	11.78	26.36
CFR 1	1000	Q500	46800	16.57	2.89	3981.69	0.040	314.79	15.06	12.53	36.16
CFR 1	700	Q500	46800	16.57	2.89	3981.51	0.040	314.78	15.06	12.53	36.16
CFR 1	400	Q500	46800	16.57	2.89	3981.34	0.040	314.78	15.06	12.53	36.17
BFR 1	1000	Q500	24900	12.57	2.11	2351.03	0.037	231.87	11.37	11.05	23.28
BFR 1	700	Q500	24900	12.57	2.11	2351.37	0.037	231.87	11.37	11.04	23.27
BFR 1	400	Q500	24900	12.57	2.11	2351.33	0.037	231.87	11.37	11.04	23.27

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Station (ft)
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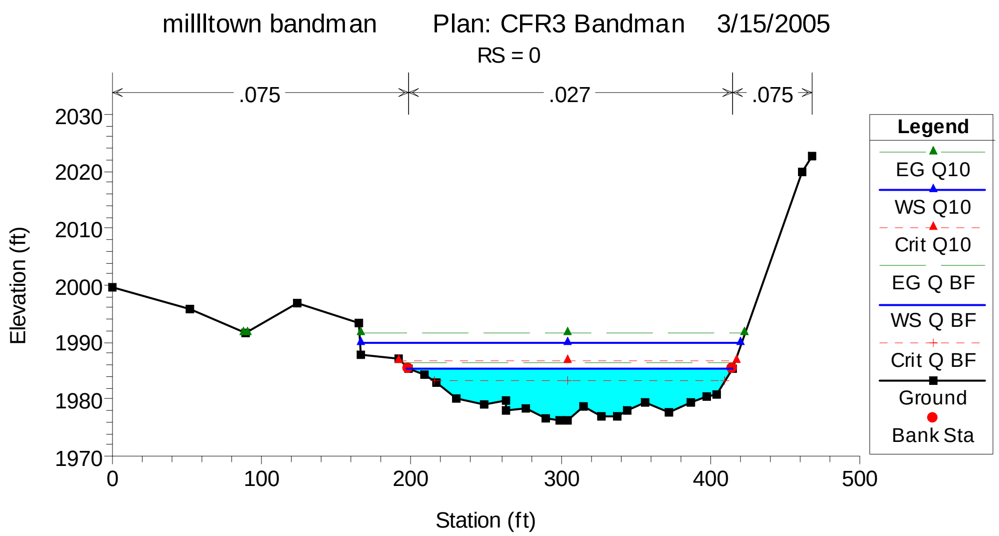
EG Q BF
WS Q BF
Ground
Bank Sta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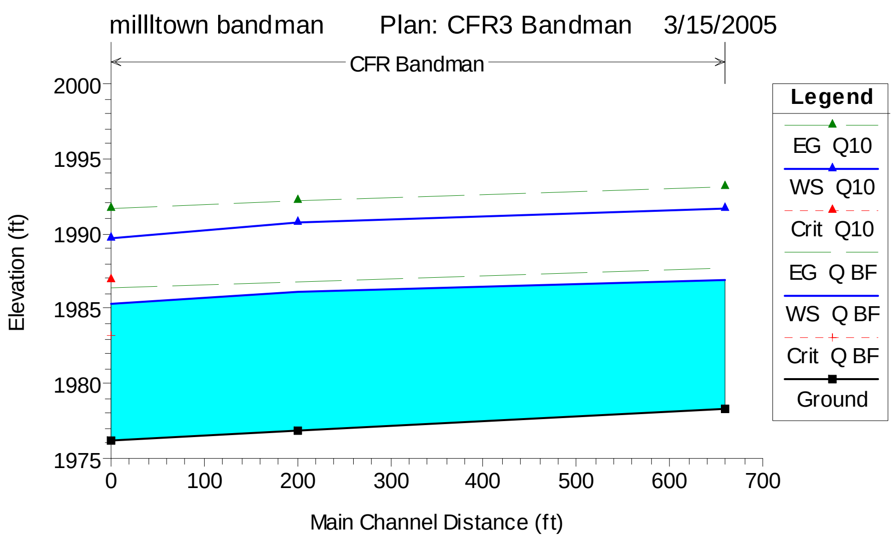
Bank Sta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CFR Bandman
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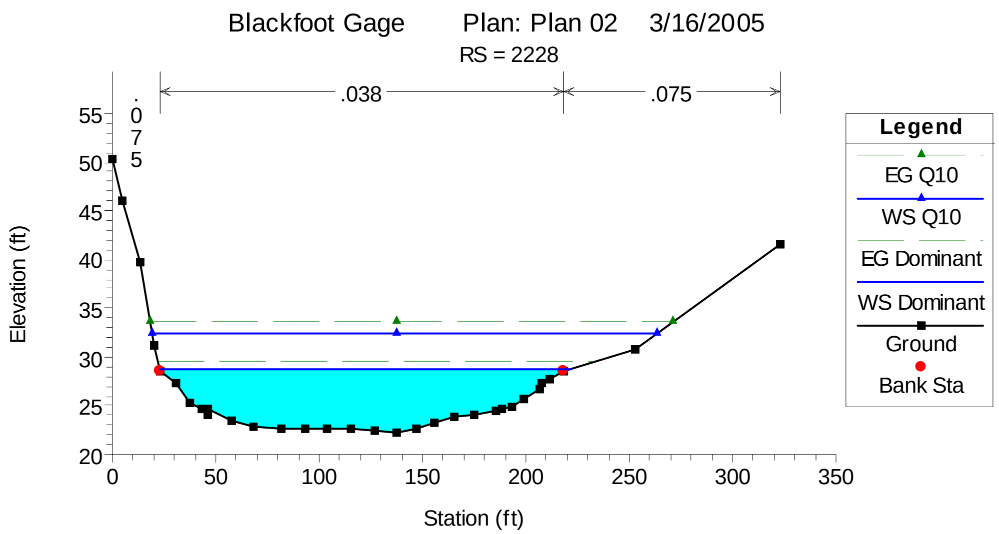
WS Q BF
Crit Q BF
Ground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Station (ft)
Legend
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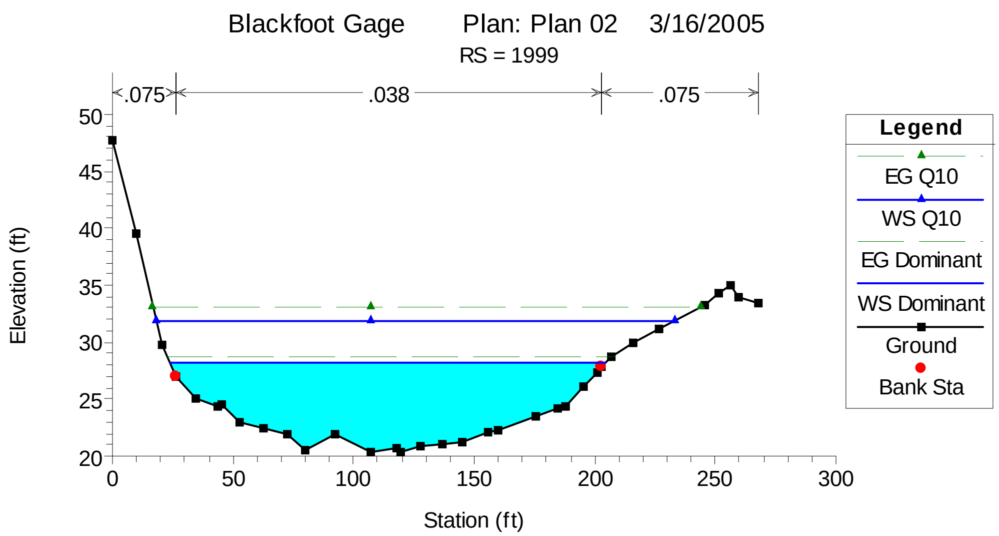
WS Q10
EG Dominant
WS Dominant
Ground
Bank Sta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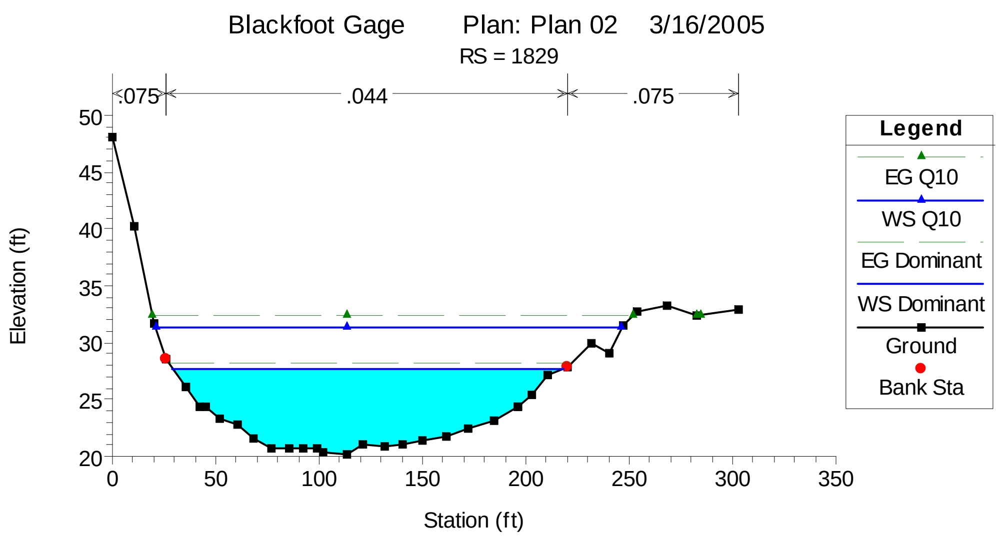
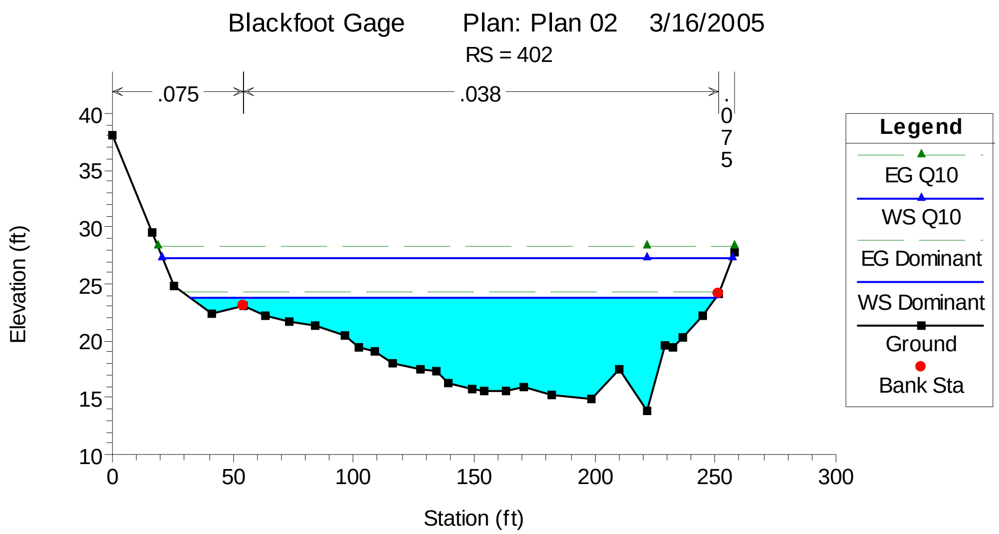
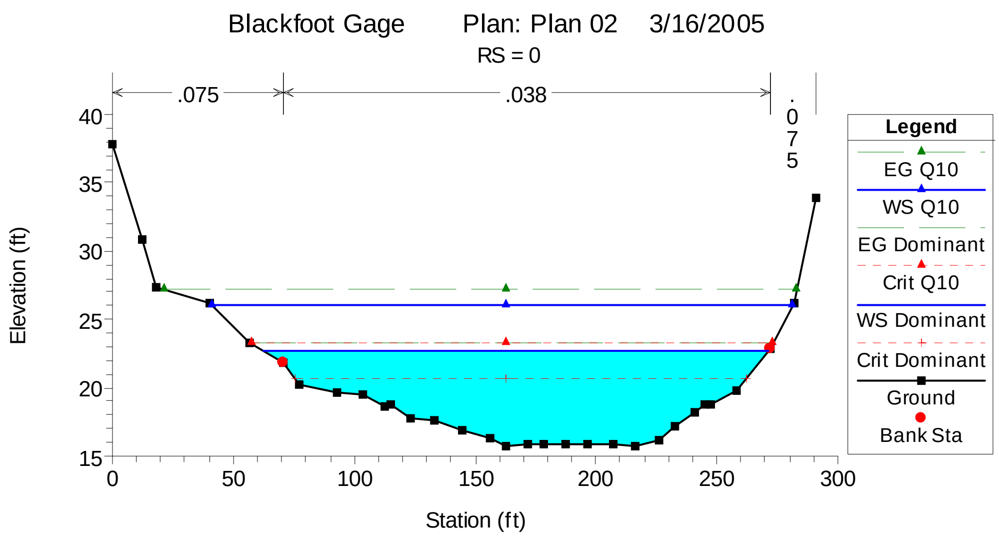
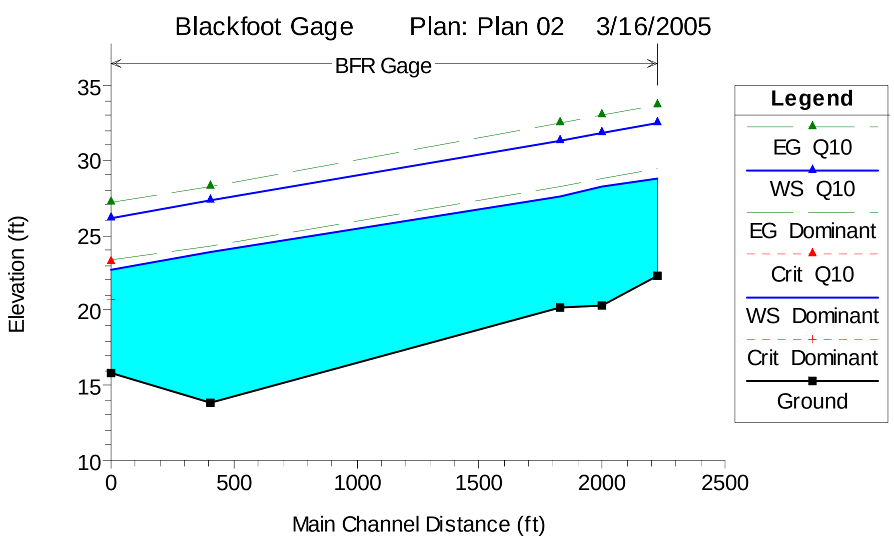
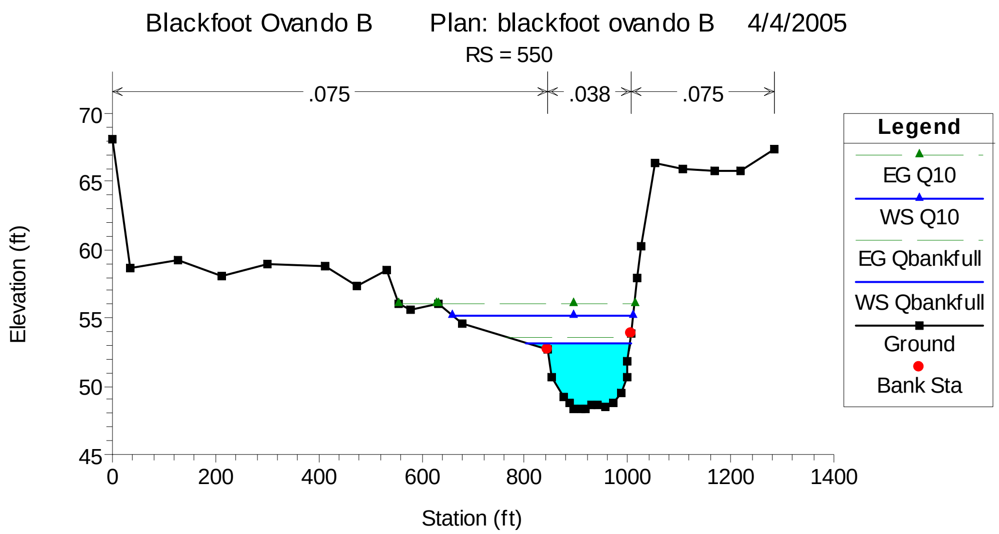
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Dominant
WS Dominant
Ground
Bank Sta

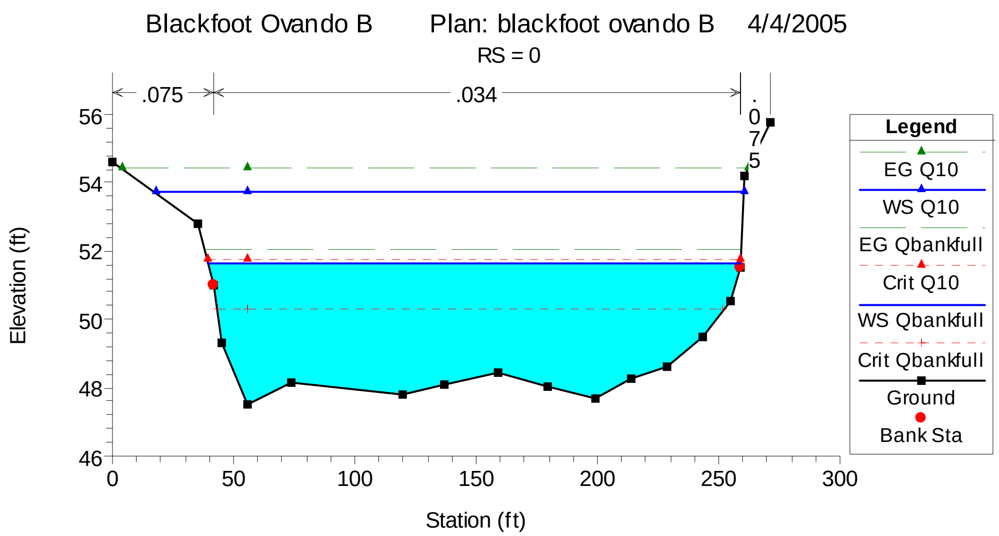

Blackfoot Gage Plan: Plan 02 3/16/2005
RS = 1829
.075
.044
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Dominant
WS Dominant
Ground
Bank Sta

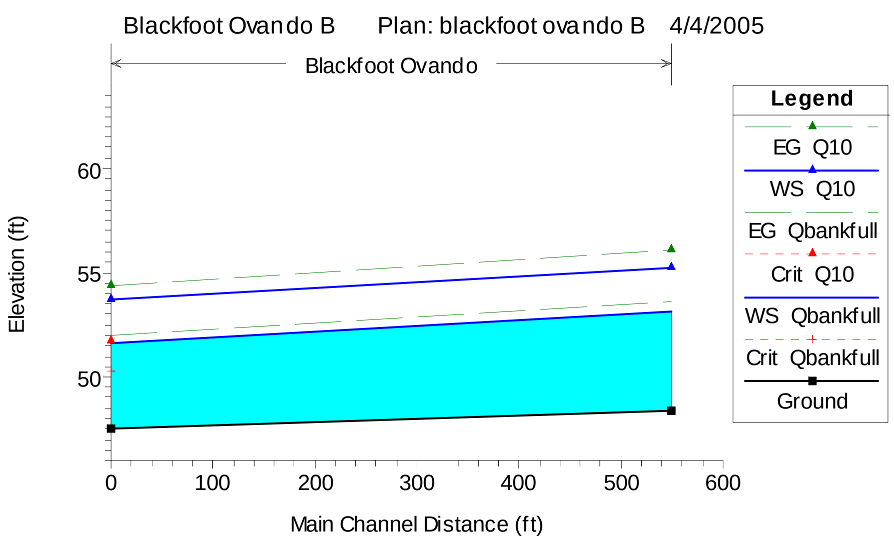

Blackfoot Gage Plan: Plan 02 3/16/2005
RS = 402
.075
.038
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Dominant
WS Dominant
Ground
Bank Sta

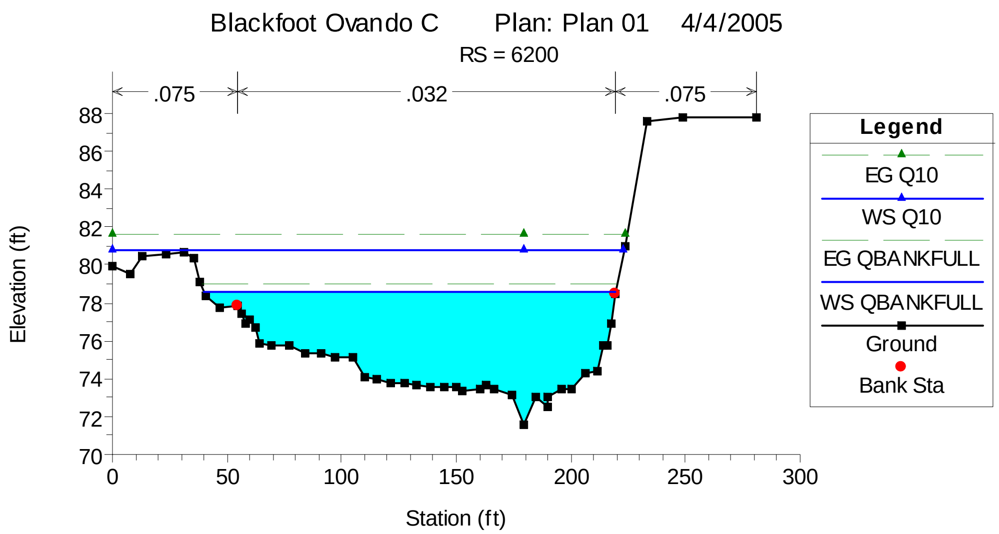

Blackfoot Gage Plan: Plan 02 3/16/2005
RS = 0
.075
.038
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Dominant
Crit Q10
WS Dominant
Crit Dominant
Ground
Bank Sta

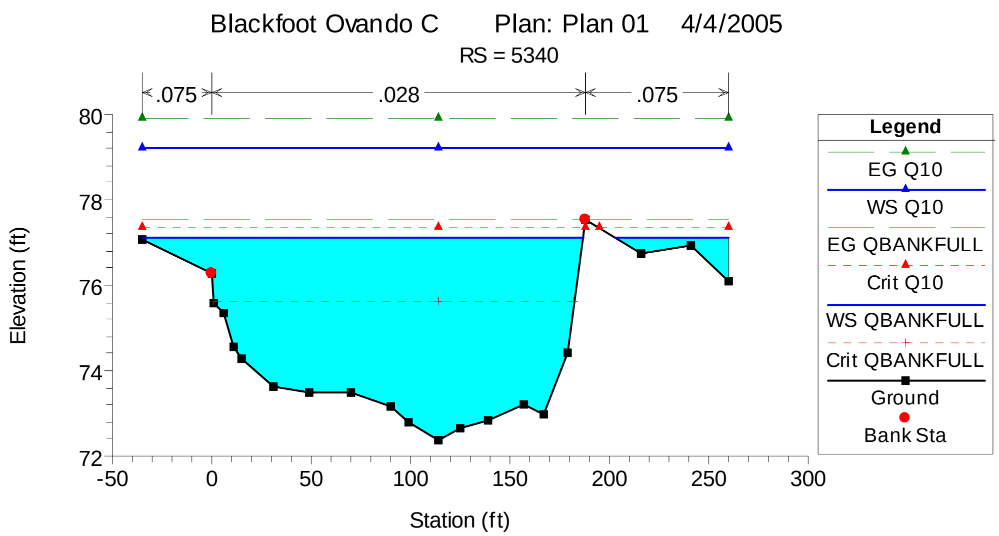

Blackfoot Gage Plan: Plan 02 3/16/2005
BFR Gage
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
EG Dominant
Crit Q10
WS Dominant
Crit Dominant
Ground

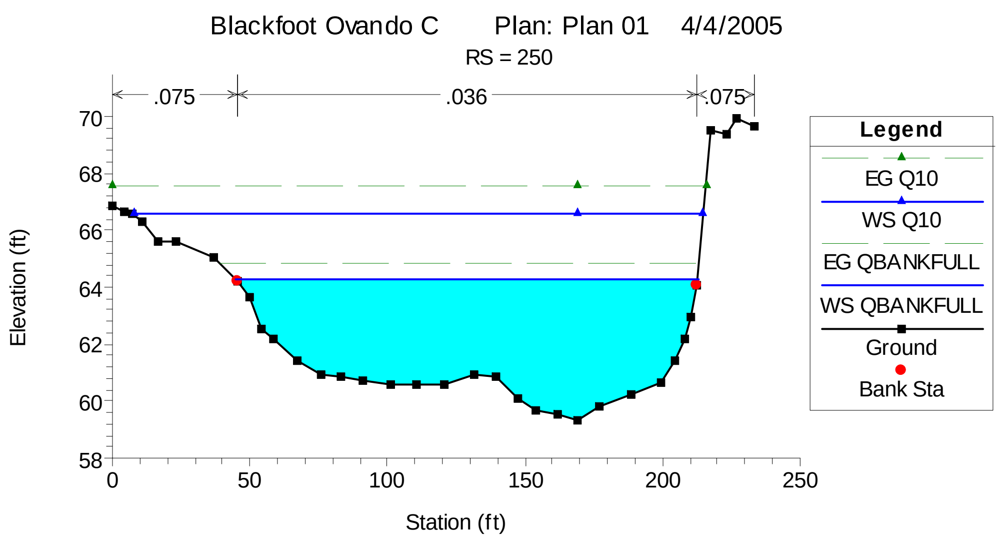

Blackfoot Ovando B Plan: blackfoot ovando B 4/4/2005
RS = 550
.075
.038
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Qbankfull
WS Qbankfull
Ground
Bank Sta

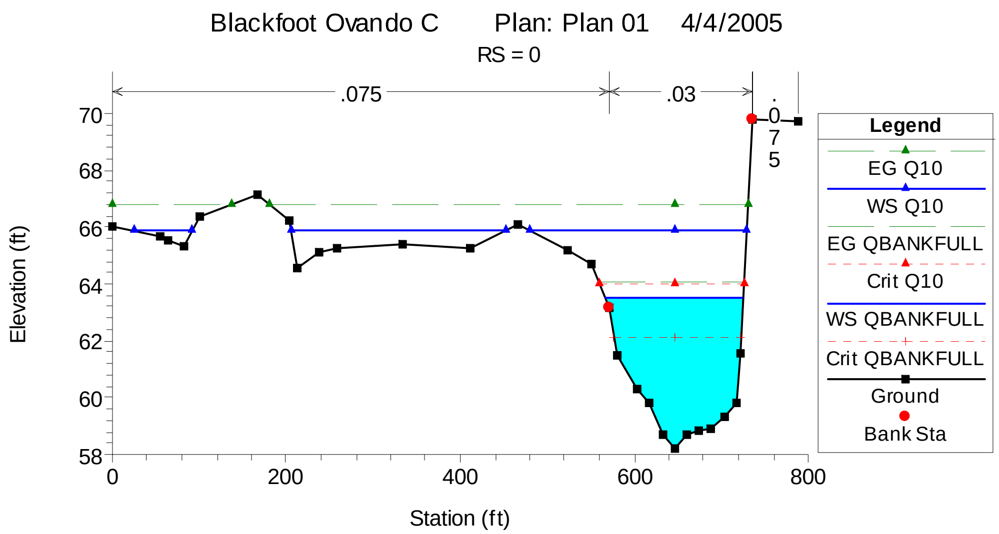

Blackfoot Ovando B Plan: blackfoot ovando B 4/4/2005
RS = 0
.075
.034
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Qbankfull
Crit Q10
WS Qbankfull
Crit Qbankfull
Ground
Bank Sta

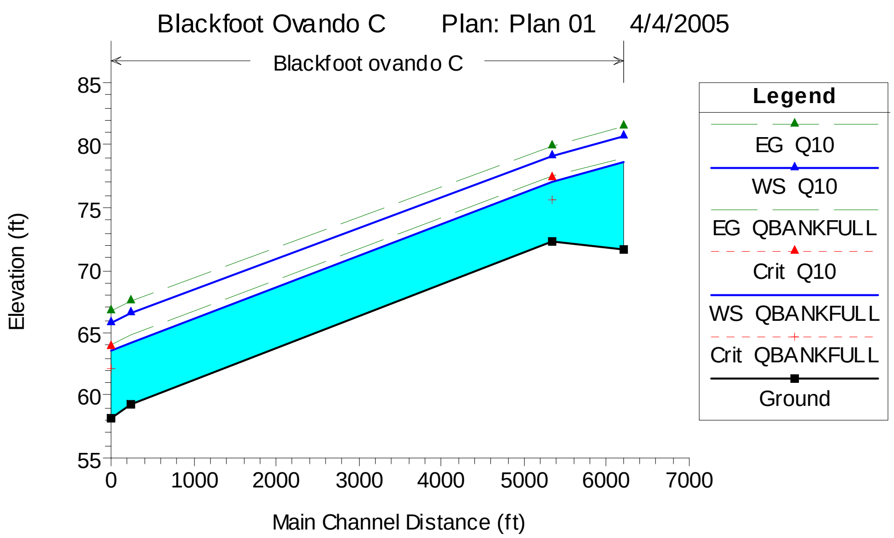

Blackfoot Ovando B Plan: blackfoot ovando B 4/4/2005
Blackfoot Ovando
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
EG Qbankfull
Crit Q10
WS Qbankfull
Crit Qbankfull
Ground

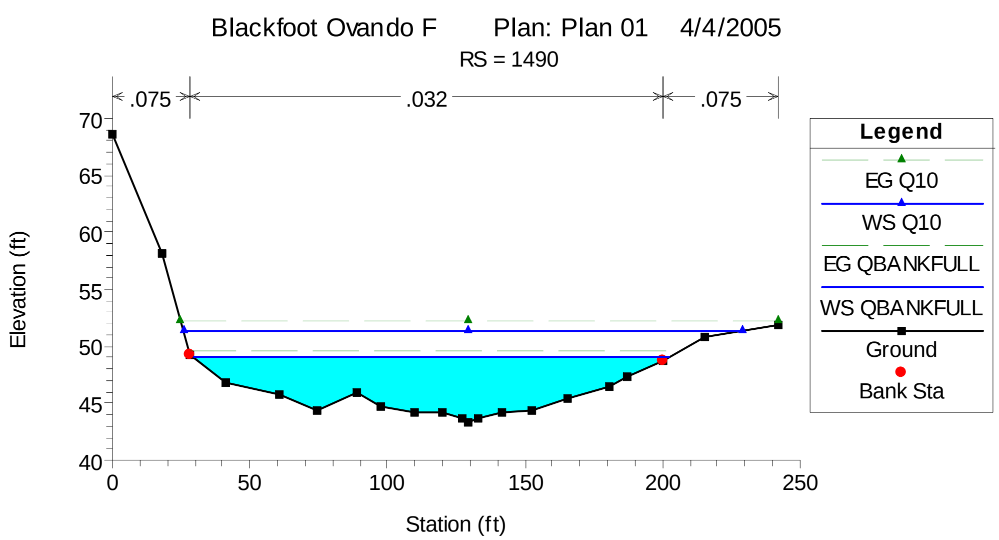

Blackfoot Ovando C Plan: Plan 01 4/4/2005
RS = 6200
.075
.032
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
WS QBANKFULL
Ground
Bank Sta

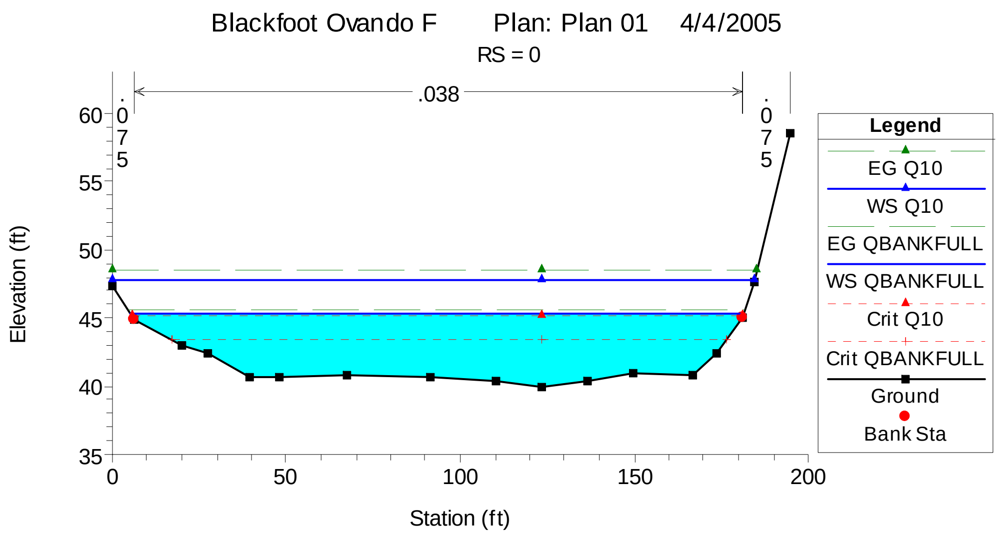

Blackfoot Ovando C Plan: Plan 01 4/4/2005
RS = 5340
.075
.028
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
Crit Q10
WS QBANKFULL
Crit QBANKFULL
Ground
Bank Sta

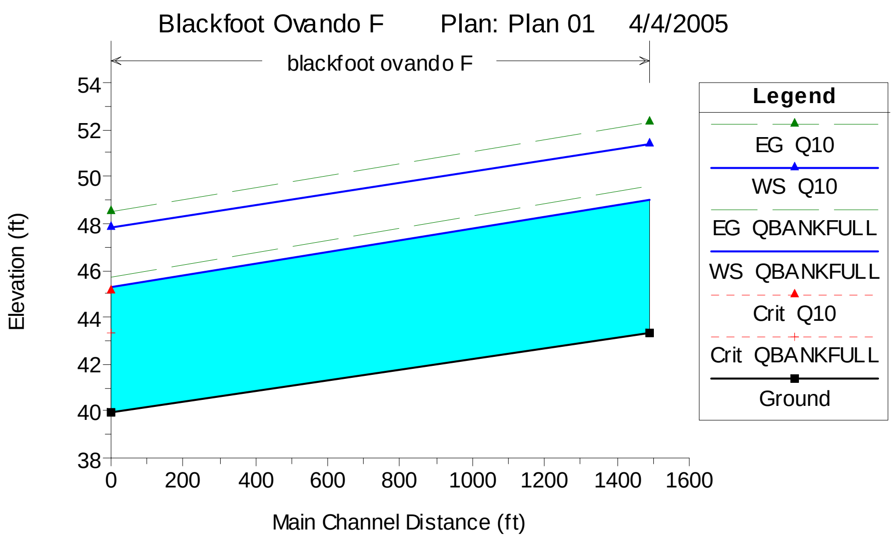

Blackfoot Ovando C Plan: Plan 01 4/4/2005
RS = 250
.075
.036
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
WS QBANKFULL
Ground
Bank Sta

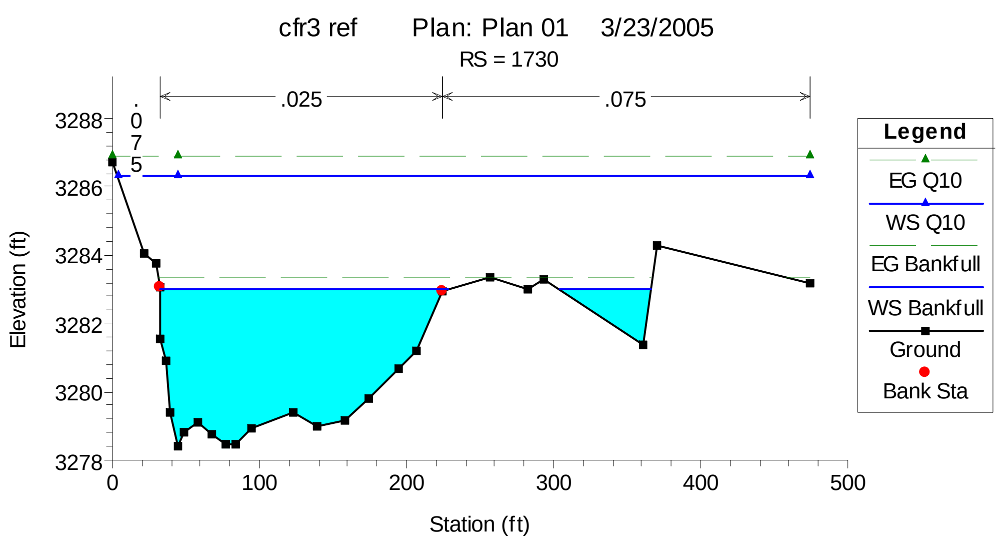

Blackfoot Ovando C Plan: Plan 01 4/4/2005
RS = 0
.075
.03
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
Crit Q10
WS QBANKFULL
Crit QBANKFULL
Ground
Bank Sta

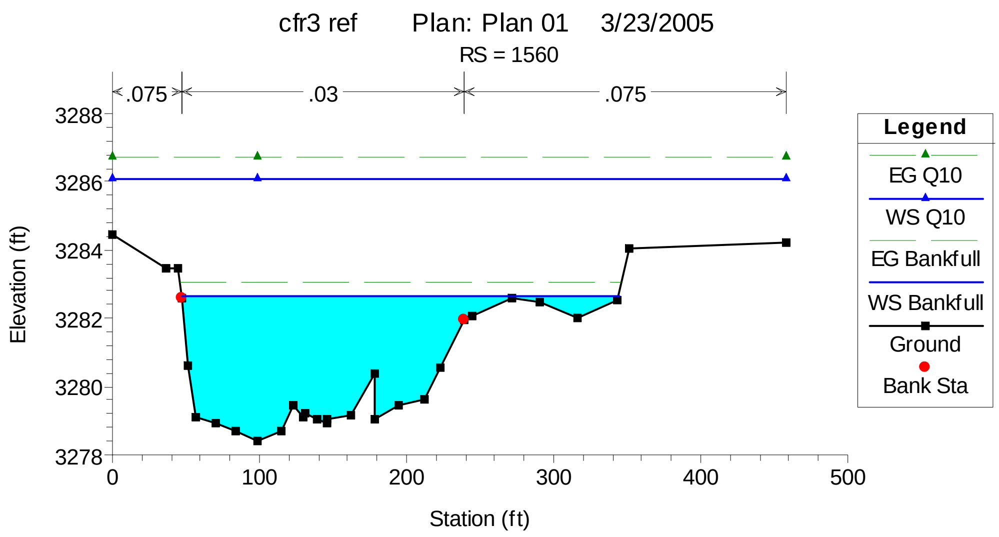

Blackfoot Ovando C Plan: Plan 01 4/4/2005
Blackfoot ovando C
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
Crit Q10
WS QBANKFULL
Crit QBANKFULL
Ground

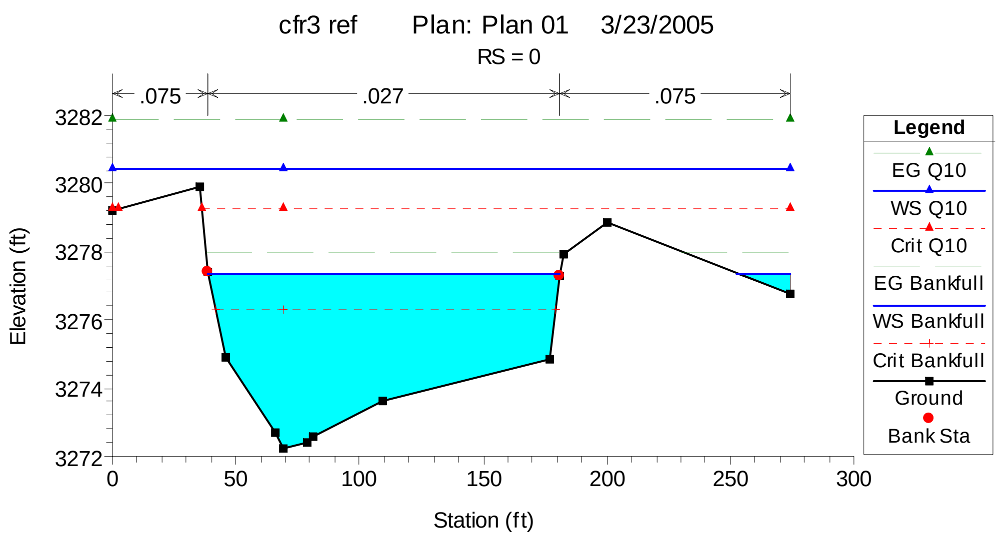

Blackfoot Ovando F Plan: Plan 01 4/4/2005
RS = 1490
.075
.032
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBANKFULL
WS QBANKFULL
Ground
Bank Sta

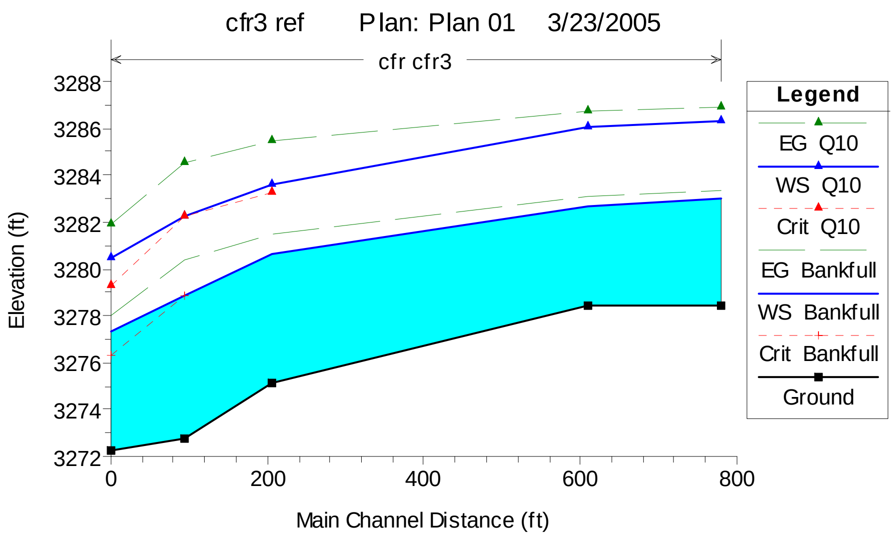

Blackfoot Ovando F Plan: Plan 01 4/4/2005
RS = 0
.038
.075
.075
Legend
EG Q10
WS Q10
EG QBANKFULL
WS QBANKFULL
Crit Q10
Crit QBANKFULL
Ground
Bank Sta
Elevation (ft)
Station (ft)
60
55
50
45
40
35
0
50
100
150
200

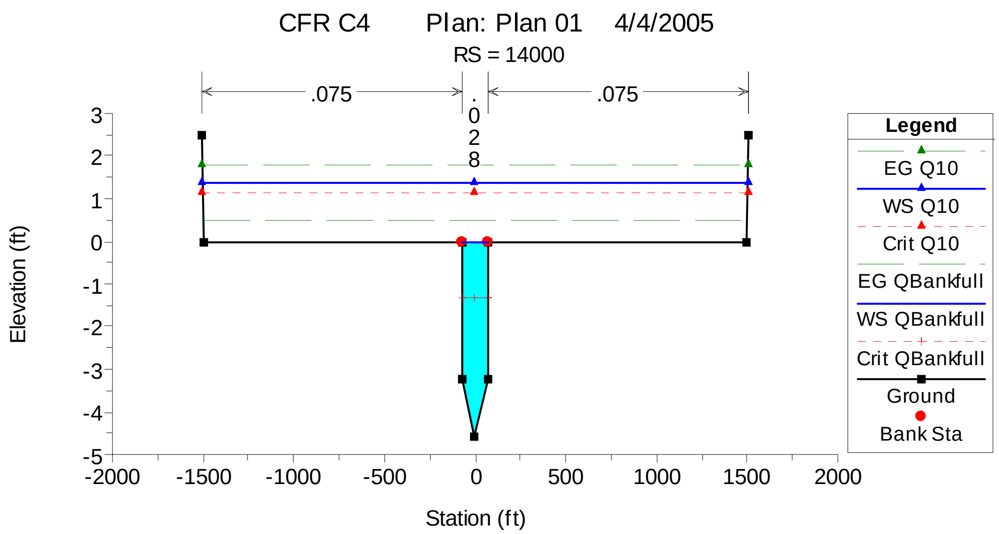

Blackfoot Ovando F Plan: Plan 01 4/4/2005
blackfoot ovando F
Legend
EG Q10
WS Q10
EG QBANKFULL
WS QBANKFULL
Crit Q10
Crit QBANKFULL
Ground
Elevation (ft)
Main Channel Distance (ft)
54
52
50
48
46
44
42
40
38
0
200
400
600
800
1000
1200
1400
1600

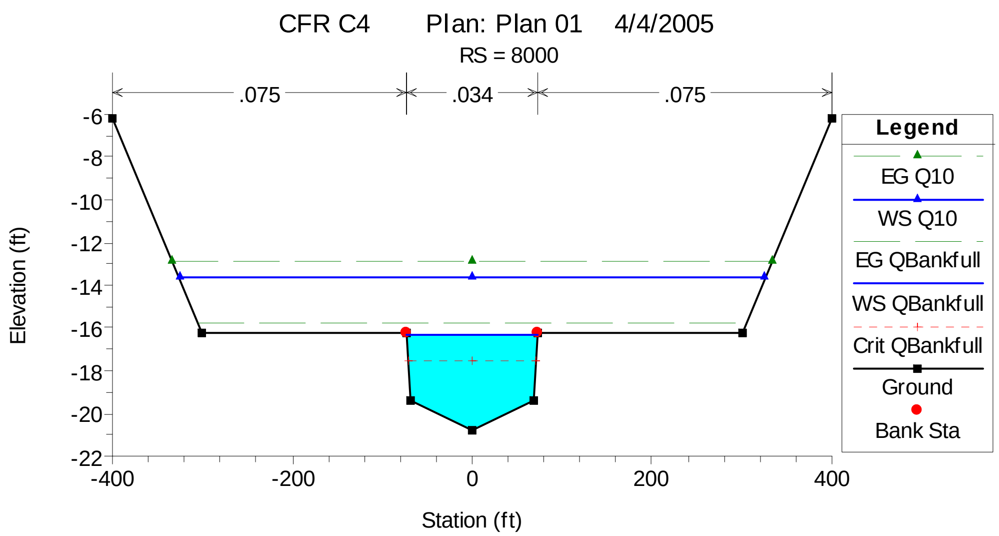

cfr3 ref Plan: Plan 01 3/23/2005
RS = 1730
.075
.025
.075
Legend
EG Q10
WS Q10
EG Bankfull
WS Bankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)
3288
3286
3284
3282
3280
3278
0
100
200
300
400
500

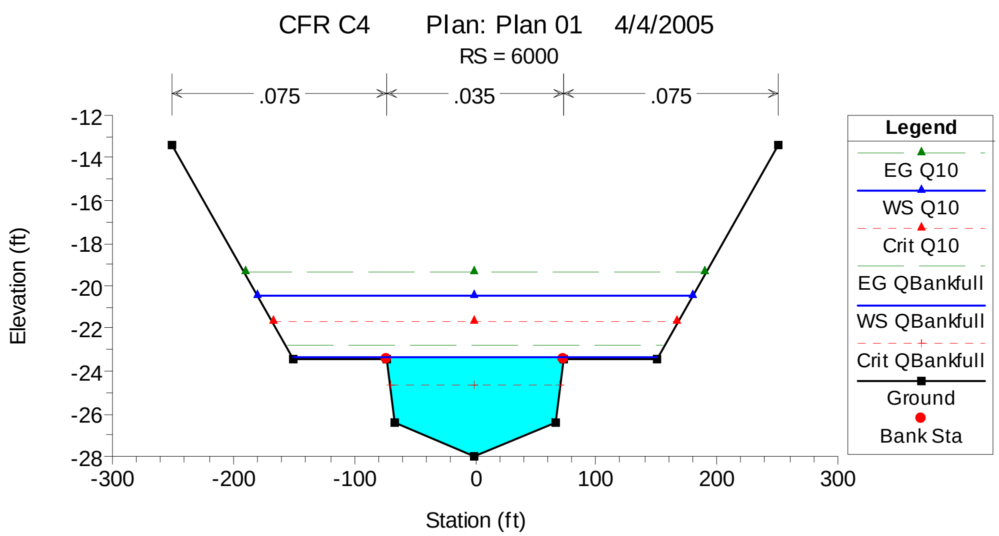

cfr3 ref Plan: Plan 01 3/23/2005
RS = 1560
.075
.03
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG Bankfull
WS Bankfull
Ground
Bank Sta

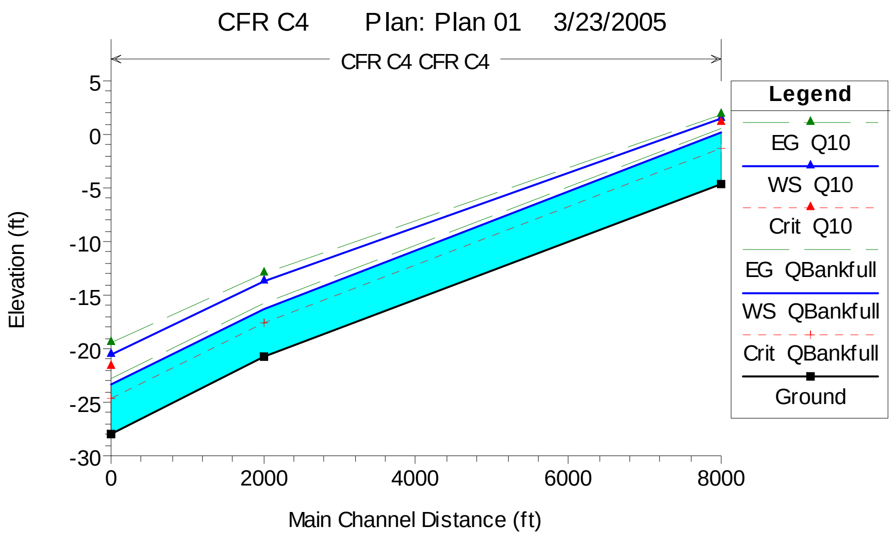

cfr3 ref Plan: Plan 01 3/23/2005
RS = 0
.075
.027
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
Crit Q10
EG Bankfull
WS Bankfull
Crit Bankfull
Ground
Bank Sta

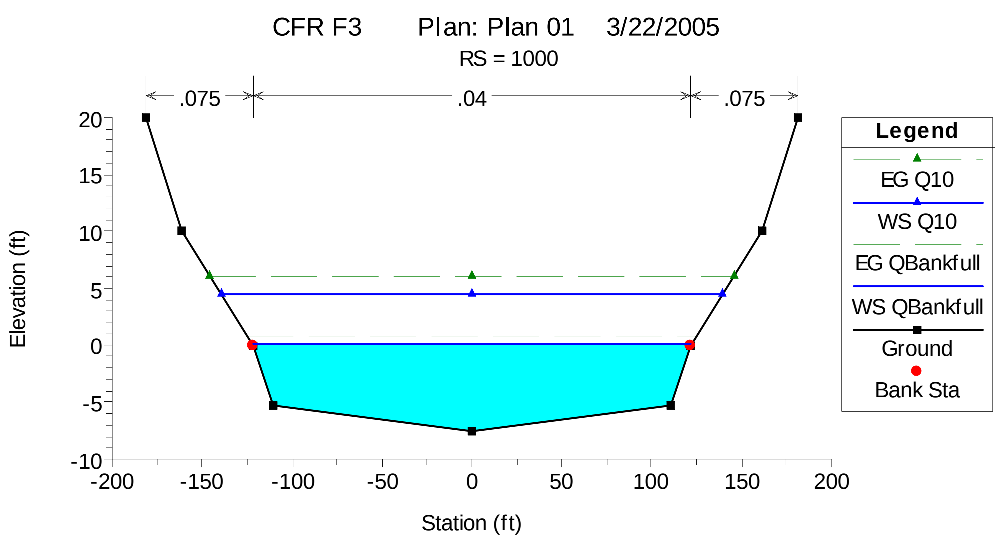

cfr3 ref Plan: Plan 01 3/23/2005
cfr cfr3
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
Crit Q10
EG Bankfull
WS Bankfull
Crit Bankfull
Ground

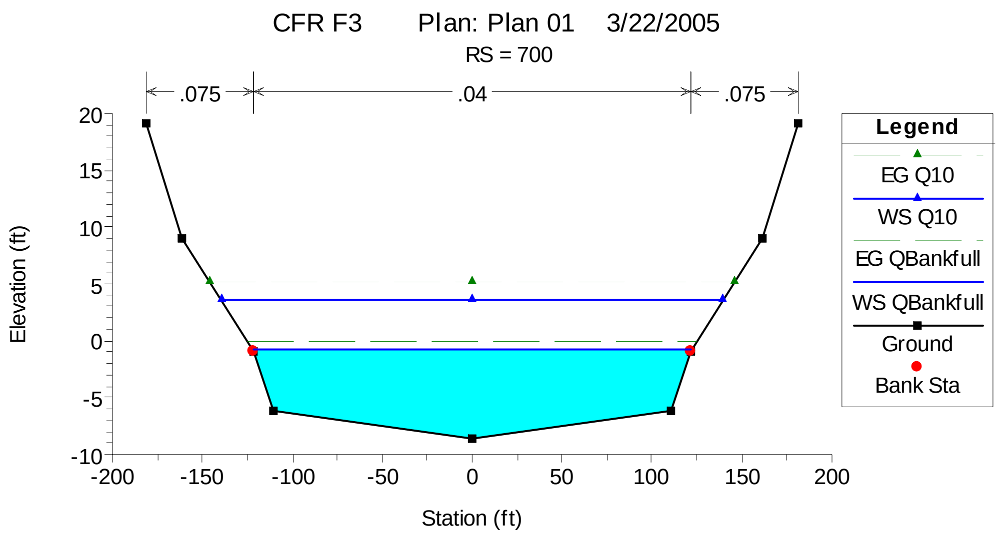

CFR C4 Plan: Plan 01 4/4/2005
RS = 14000
.075
.028
.075
Legend
EG Q10
WS Q10
Crit Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)

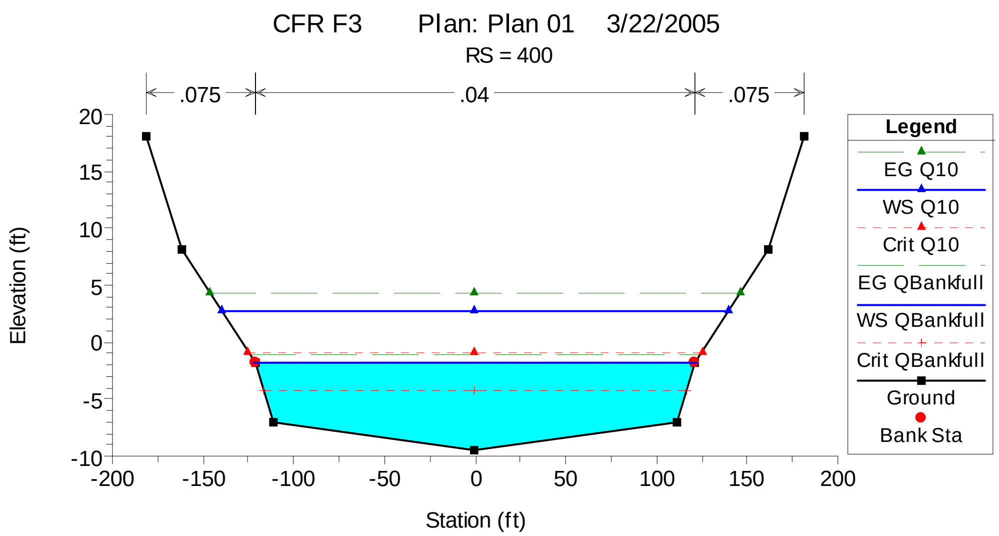

CFR C4 Plan: Plan 01 4/4/2005
RS = 8000
.075
.034
.075
Legend
EG Q10
WS Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)

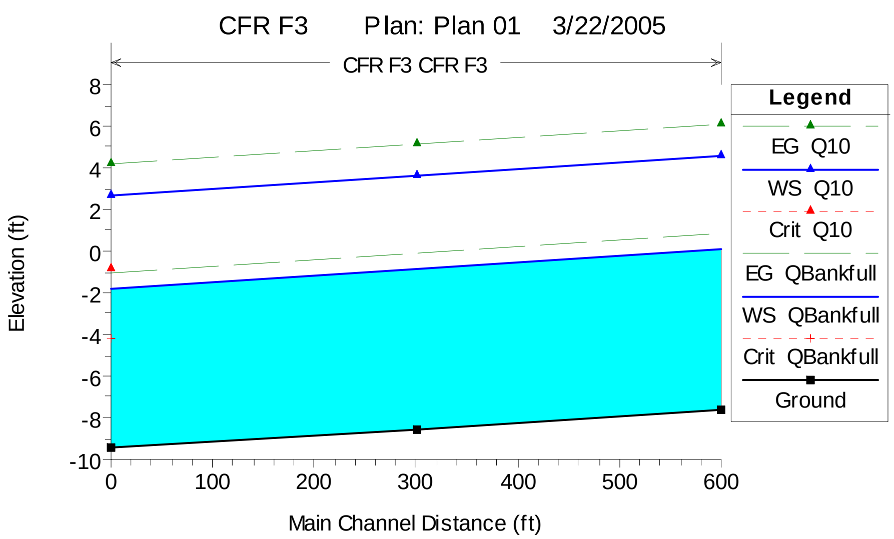

CFR C4 Plan: Plan 01 4/4/2005
RS = 6000
.075
.035
.075
Legend
EG Q10
WS Q10
Crit Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)

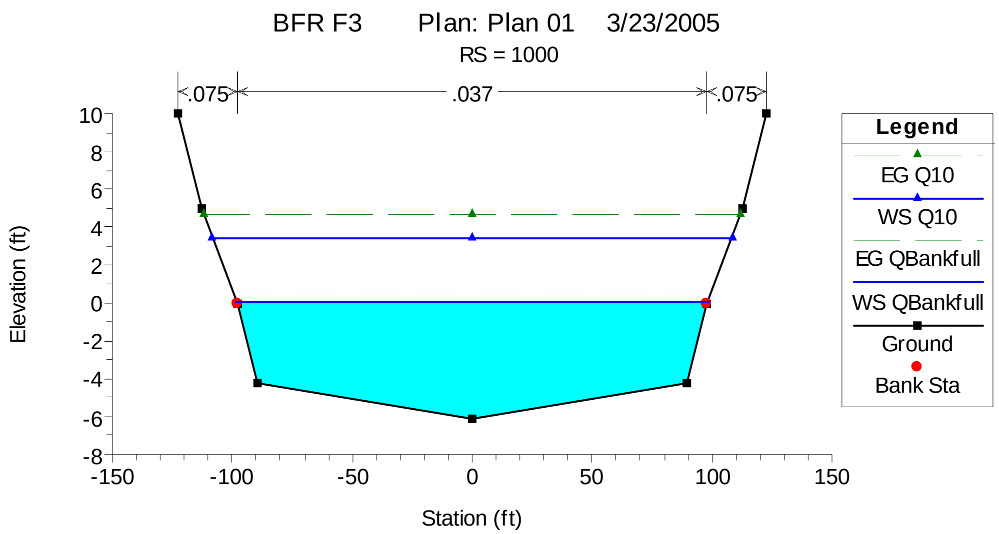

CFR C4 Plan: Plan 01 3/23/2005
CFR C4 CFR C4
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
Crit Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground

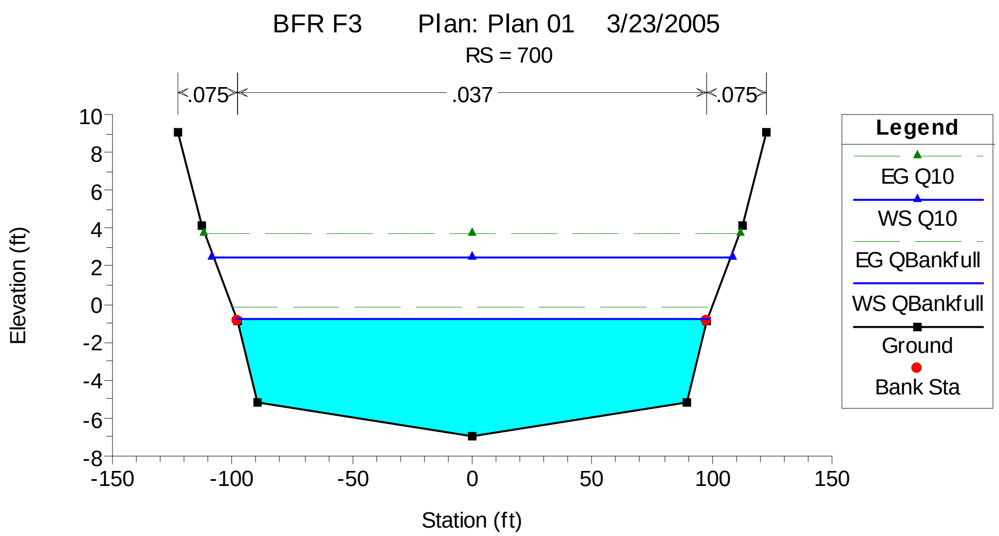

CFR F3 Plan: Plan 01 3/22/2005
RS = 1000
.075
.04
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBankfull
WS QBankfull
Ground
Bank Sta

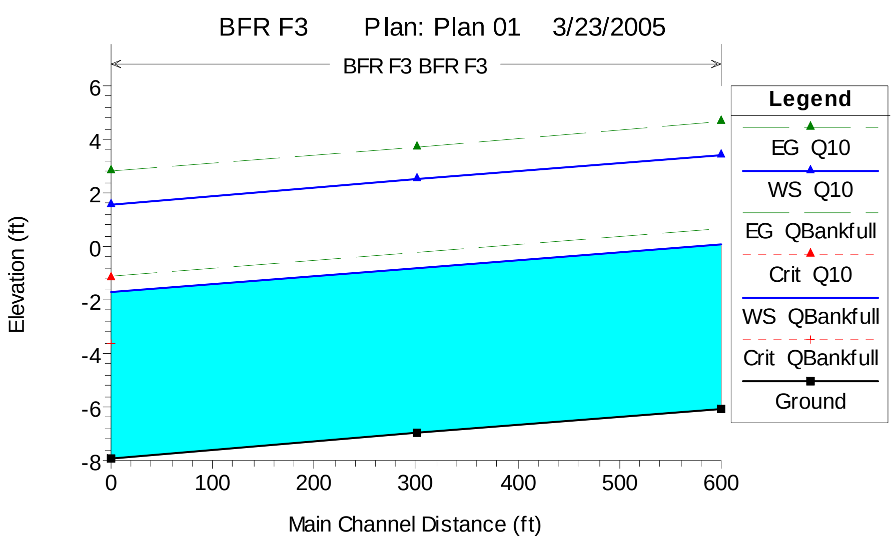

CFR F3 Plan: Plan 01 3/22/2005
RS = 700
.075
.04
.075
Legend
EG Q10
WS Q10
EG QBankfull
WS QBankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)

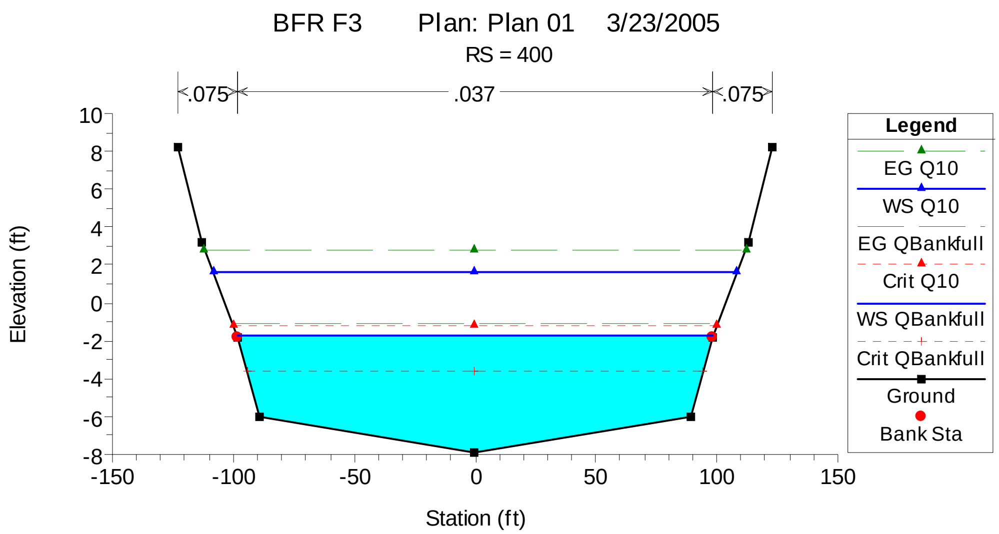

CFR F3 Plan: Plan 01 3/22/2005
RS = 400
.075
.04
.075
Legend
EG Q10
WS Q10
Crit Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground
Bank Sta
Elevation (ft)
Station (ft)


CFR F3 Plan: Plan 01 3/22/2005
CFR F3 CFR F3
Legend
EG Q10
WS Q10
Crit Q10
EG QBankfull
WS QBankfull
Crit QBankfull
Ground
Elevation (ft)
Main Channel Distance (ft)


BFR F3 Plan: Plan 01 3/23/2005
RS = 1000
.075
.037
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBankfull
WS QBankfull
Ground
Bank Sta


BFR F3 Plan: Plan 01 3/23/2005
RS = 700
.075
.037
.075
Elevation (ft)
Station (ft)
Legend
EG Q10
WS Q10
EG QBankfull
WS QBankfull
Ground
Bank Sta


BFR F3 Plan: Plan 01 3/23/2005
BFR F3 BFR F3
Elevation (ft)
Main Channel Distance (ft)
Legend
EG Q10
WS Q10
EG QBankfull
Crit Q10
WS QBankfull
Crit QBankfull
Ground


BFR F3 Plan: Plan 01 3/23/2005
RS = 400
.075
.037
.075
Elevation (ft)
Station (ft)
10
8
6
4
2
0
-2
-4
-6
-8
-150
-100
-50
0
50
100
150
Legend
EG Q10
WS Q10
EG QBankfull
Crit Q10
WS QBankfull
Crit QBankfull
Ground
Bank Sta